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In this issue

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LIST OF ADVERTISERS

Directory of Professional Services—pp. 25-29

COMING MEETINGS

November 13–15—North Carolina Section at Washington-Duke Hotel, Durham, N.C. Secretary: E. C. Hubbard, Prin. San. Engr., State Board of Health, Raleigh, N.C.

13–15—Joint Cuban-Florida Water and Sewage Conference (Cuban and Florida Sections A.W.W.A. and Florida Sewage Works Assn.) at Governor's Club Hotel, Fort Lauderdale, Fla.

Nov. 30–Dec. 2—Arizona Section at Pioneer Hotel, Tucson, Ariz. Secretary: Mrs. Helen Rotthaus, San. Eng. Div., State Dept. of Health, Phoenix, Ariz.

January 16, 1951—New York Section Luncheon Meeting at Park-Sheraton Hotel, New York, N.Y. Secretary: R. K. Blanchard, Vice-Pres. & Engr., Neptune Meter Co., 50 West 50th St., New York 20, N.Y.

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Contents

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Journal

AMERICAN WATER WORKS ASSOCIATION

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Basic Considerations in Determining Water Rates

By Louis E. Ayres

A paper presented on May 25, 1950, at the Annual Conference, Philadelphia, by Louis E. Ayres, Civ. Engr., Ayres, Lewis, Norris & May, Ann Arbor, Mich.

THIS paper presents the *tentative* conclusions of the author, as chairman of the Committee on Water Rates, on certain basic considerations involved in determining water rates, as outlined to the committee on March 14, 1949, presented to the Association in a panel discussion (1) at the 1949 Annual Conference and subsequently commented on in letters from committee members to the chairman.

These basic problems have been listed (1) as follows:

1. How much revenue should a water works receive?
2. In what proportions should this gross revenue be collected: (a) from property assessments and (b) through utility rates?
3. Should utility rates incorporate a "demand" or "ready-to-serve" charge; and, if so, upon what logical basis should the amount of such a charge be fixed?
4. If a "demand" charge is to be used, upon what basis should it be distributed to customers?
5. Finally, how many steps are desirable in the commodity rate; and what are the logical rules for fixing the step quantity limitations and the unit rate for each step?

Amount of Revenue Needed

Public and Private Ownership

Should there be a difference in the approach to the subject of revenue as between privately owned and publicly owned water works? It was originally the author's thought that there was a difference and that the committee discussions would be concerned largely with publicly owned properties. It was premised that all privately owned, but only a relatively few publicly owned, plants are subject to the rulings of courts and commissions; and that the A.S.C.E.-sponsored Joint Committee on Fundamental Considerations in Rates and Rate Structures for Water

and Sewage Works would survey and compile the pertinent information on court and commission decisions.

It has developed however, that certain committee members believe the approach to the subject should be the same whether a water works is privately or publicly owned. As one committee member put it, the only essential difference is in the rate of return on the used property. He proposed "for interest, if a private company, from 5 to 7 per cent, dependent upon location and cost of money"; and "for interest, if a publicly owned system, from 2 to 4 per cent, which includes the interest on outstanding bonds, if any."

There is much precedent for the assumption that there should be no difference between a privately and publicly owned water works as regards the amount of return which should be received. The Wisconsin Public Service Commission is required by statute to adopt the basic principle that: "A municipally operated water utility is *entitled to the same rate of return* as that permitted for privately owned utilities" (2).

Allen Hazen (3) took the position many years ago that "the revenue produced must be sufficient to pay all operating expenses of every description, including the general administration, and this includes, in the case of works owned by cities, a proportionate part of the general expenses of the city government and of the city buildings, a fair allowance for depreciation on all property used, all taxes of every kind, and in addition, a return on the value of the plant at the rate at which money can be borrowed for the enterprise." And he goes on to declare that such a basis "if systematically followed, will result in constantly increasing surplus revenue, and ultimately part of the surplus should be dedicated to other nonproductive public purposes, such as supplying libraries, hospitals and parks."

Although there may be no illustrations of the use of water works surplus for the financing of "libraries, hospitals and parks," it is a fairly common practice for water works moneys to be turned over to the general fund of the city. In the A.W.W.A. tabulation of water works data for 1945 (4), about 170 cities of a total of 462—37 per cent—contributed to the general fund in varying amounts in 1945. Some of these cities turned over all of their receipts. Others turned over net earnings less debt requirements and, sometimes, less payments to a reserve. In addition, about an equal number, although not the same cities, gave free service. Some cities reported no allowance for depreciation, while one had a depreciation charge equal to 16.5 per cent of book value. Many paid no taxes and received no compensation for fire protection.

From other, unpublished reports, the author notes that 7 of 36 of the nation's largest cities paid all "excess revenue over expense" to the general fund, and one city paid a "dividend on stock owned by the city."

A review of available data presents a confusing picture of the financial affairs of many municipally owned water works. From spot inspection of the record, it appears that the return on book value varies widely, ranging from practically nothing to upwards of 20 per cent in locally controlled plants. In plants under commission control, the rate of return is limited.

The higher rates of return in locally controlled plants are due to [1] the laying aside of portions of net revenue

for future improvements and [2] the diversion of water works funds to meet deficiencies resulting from other municipal expenditures. But in state-controlled plants, commission rulings appear to deny the right of cities to raise, through increased water rates, the revenue necessary to finance future improvements. As an example, the Montana commission (re *City of Helena*) recently held that "the future sale of bonds does not warrant *present* increases to provide for principal and interest payments to commence two or three years hence." Regardless of one's viewpoint on the propriety of diversions to the general fund, it can be agreed that such diversions are to be deprecated unless and until full provision has been made for all of the legitimate needs of the water works, both immediate and future.

Two means of reducing existing confusion suggest themselves: [1] placing all water works under the supervision of state commissions; or [2] promulgating rules of financial conduct which will permit the exercise of local preferences, yet lead to ample financial support for an adequate water works.

State supervision is never likely to be accepted throughout the nation. It has long been a controversial matter, opposed by those who regard it as an unnecessary interference in the right of cities to manage their own affairs. As of August 5, 1948, only five state commissions—Montana, Nevada, West Virginia, Wisconsin and Wyoming—were reported to have jurisdiction over municipally owned water works in the regulation of rates within the corporate limits.

If, in lieu of state supervision, regulations are to be drafted for the "home rule" of water works, they must depart from the general premise of state commissions that, in the approach to the matter of revenue, there is no difference between publicly and privately owned systems.

That there are very definite differences may be seen from the following:

1. Privately owned water works, in order to survive, must receive a rate of return which will assure that new money for extensions and improvements can be obtained at interest rates attractive to investors. There are no such limitations on the financing of publicly owned properties. Money is borrowed, usually on either general-obligation or revenue bonds. The general-obligation bond is a lien on all the property of the city; its salability is not affected by any consideration of the assets or earnings of the water department. Revenue bonds are a lien on the earnings of the water works, but the buyer is interested mainly in the net income and the assurance that the principal and interest on his particular bond issue will be paid out of revenue. He is not primarily concerned with the value of the debt-free property or with whether the city pays any return on such property or not. But the rate in effect must be sufficient, with a safety margin, to pay all the cash obligations of the system.

2. In the privately owned property, bonds are never retired; they are refunded. In the publicly owned system, bonds are generally retired. The private company's earnings on debt-free property are available as stock dividends; with the public system, the debt-free property is owned by the city. Who owns this property, in equity—the taxpayers or the water rate payers—is another moot question; but the decision on whether or not it should pay

interest, and to whom, may be considered a local problem to be determined by local authorities.

3. The difficulty in applying the theory that the municipally operated water utility is "entitled to the same rate of return as that permitted for privately owned utilities" is that *one must establish a value for the property and determine the interest rate.* In some utilities, the undepreciated book value is used, but such an amount is likely to be much less than the value allowed by a court on a privately owned utility. For privately owned utilities, these matters are determined by court and commission adjudication; for locally controlled, publicly owned utilities, no such machinery is available. Moreover, the criteria are not the same for both. A return of 5 per cent, for example, may suffice to maintain a solvent private company, but a similar return on a publicly owned property may be less than or much more than needed, depending upon the value of debt-free property and the policies regarding major improvements and other items.

Basis for Public Systems

In publicly owned plants, a practical method is to approach the problem from the standpoint of the *cash* needs, based on adequate estimates of operation and maintenance costs, replacement allowances, and actual interest and bond retirements, together with such amounts as may be found necessary for extensions and improvements. With the total thus determined, one may compute the interest rate required on the book value or appraised value of the property, but this interest rate is not necessarily significant. For the average municipal water works, owning much debt-free property, the interest rate may be relatively low, unless major extensions are being financed at a rapid rate, in which event it may be relatively high. If the high rate is due to fund diversions, that is another matter, which is open to serious question.

Such a budget or cash basis for the determination of gross revenue for publicly owned water works seems to the author to offer a realistic approach to the problem, one which will preserve and defend the water works, yet leave matters of policy to the officials and citizens.

On this basis, the revenue requirements may be considered under two classes: *basic* and *optional*:

Basic

1. Operation and maintenance
2. Debt payments
3. Reserves for:
   a. Replacements
   b. Normal extensions and improvements

Optional

4. Taxes
5. Reserves for major improvements
6. Contributions to other city departments

These items may be described briefly, as follows:

1. *Operation and maintenance.* There should be *adequate* provision for efficient operation and maintenance of the property. This means that the organization should have sufficient, trained personnel not only for bare operation, but to provide competent supervision of office books, records, accounting, and maps and planning. The wage and salary schedule must enable the utility to retain its personnel against the competition of business and industry.

2. *Debt payments.* The revenue should suffice to meet the *actual* interest

and retirement obligations on all of the outstanding bonds regardless of the security behind them. Although general-obligation and special-assessment bonds are the legal obligations of the whole or a part of the taxpayers, the payment of interest thereon and the retirement of the principal are commonly accepted as the moral obligation of the water works and as payable out of revenue, similarly to mortgage or revenue bonds.

3. *Reserves.* The word "reserves" is used in an accounting sense. Certain moneys should be taken currently from income, credited to suitable reserve accounts and charged to their corresponding funds; and the cash in these funds should be used only for the purpose for which they are created.

Basic reserves are suggested for replacements and for normal extensions. Replacement allowances should be *just sufficient* to provide for the renewals and replacements which may be anticipated from decade to decade in a property that, on the whole, will continue to be serviceable in future generations. Allowances for normal extensions should not include any major improvements. If a sinking fund for debt amortization is required, there should be a separate reserve and fund. Funds should be kept in cash or negotiable investments and the adequacy of the accumulations should be reviewed at five-year intervals.

The above items cover the *basic* requirements, with which no water works manager, council or commission would disagree. The optional or additional items involve those matters on which there is a difference of opinion and which, in the last analysis, will be decided by the local authorities:

4. *Taxes.* The opinions of committee members on this subject appear to range from no allowance for taxes to one for all of the taxes—local, state and federal—paid by a privately owned utility. There seems to be agreement, however, that taxes should not be included unless adequate payment is made to the utility for fire protection and all free service. Of the 462 cities listed in the A.W.W.A. survey (4), about 43 per cent paid taxes of some sort.

5. *Reserves for major improvements.* If it is local policy to avoid debt, in whole or in part, in meeting the cost of major system improvements, a reserve should be created for this purpose and the funds kept apart, carefully invested and earmarked, without possibility of diversion for other purposes. It is not proper accounting to collect such funds through excessive depreciation rates.

6. *Contributions to other city departments.* The most controversial item among the optional additions to revenue is the contribution of the water works to other departments of government over and above a reasonable charge for services rendered by the latter to the utility. The danger inherent in such a contribution is that, once adopted as a policy, it affords an opening wedge for further claims on the revenues of the water works to supplement the city budget. Such a procedure is, in effect, a charge on the rate payer for the benefit of the taxpayer, and its equity may be questioned. If the debt-free property has been accumulated out of receipts from the water users, how can it be considered the property of the taxpayers, upon which they are entitled to receive a return? Only to the extent that water works debt-free property has been paid for by taxpayers can one justify a "dividend" to taxpayers on such property.

Allocation of Revenue

The second item in the present outline of the Committee on Water Rates has to do with the proportions of gross revenue that should be collected (a) from taxpayers and (b) from customers through commodity rates; and these two groups might be classified also as property and persons, and as nonusers and users.

A water works provides a wider service than merely supplying a commodity. A pure water supply is important to public health; the existence of a water system enhances the value of all property which may be supplied, whether actually served or not; and the availability of water for fire protection safeguards the community and effects large savings in fire insurance.

Although public health and land benefits are real, they are intangible, and there are but few instances in which they appear to have been taken into account in collecting revenue for a water works. Fire protection, on the other hand, is a property benefit that is tangible and may be evaluated and assessed.

One outstanding example of a rate structure which recognizes property benefits in a large way is that of the Washington Suburban Sanitary Dist., where the following taxes, assessments and charges are made: [1] a general tax on *all* property, based on the assessed valuation; [2] a "front-foot benefit charge," where a water main and sewer exist in front of a lot, *whether used or not;* and [3] a "ready-to-serve charge" and a "consumption charge" when water is used (5).

To take a typical illustration, a vacant lot, with water main and sewer in the street, pays 55 per cent as much as the same lot with a house on it using 5,000 gal. of water per month. The front-foot benefit charge, for both water and sewerage, is more than $1\frac{1}{2}$ times the water consumption charge. It is interesting to note that, in this instance, "there is no limitation on the taxing authority of the district."

The theories behind such charges have yet to be developed for specific application. That is one of the tasks assumed, it is understood, by the A.S.C.E.-sponsored joint committee.

For the present, at least, the A.W.W.A. committee is confining its studies on revenue allocation to fire protection (paid by taxpayers) and commodity rates (paid by customers).

Fire Protection Charges

Although the propriety of a charge to the taxpayer for public fire protection has long been acknowledged by the industry, by its managers, boards and engineers, only a minority of municipally owned water works collect any of their revenue in this way.

Based on the A.W.W.A. survey of operating data for 1945 (4), it appears that only 193 out of a total of 462 cities, or 42 per cent, collect *any* revenue on account of fire protection. Of the 462 cities reported, 43 are privately owned and 419 publicly owned. Of those privately owned, 81.4 per cent collect a fire charge, but of the publicly owned, only 37.7 per cent. More striking than these figures, however, is the *range* of the charges made, which varies from a token amount of $0.015 to $4.20 per capita per year. Obviously, the tabulation referred to is far from complete, but it indicates a state of utter confusion.

Previous Methods

Numerous methods have been proposed in the past for determining the

proper proportion of water works costs chargeable to fire protection. Three of the methods described in the extensive literature on the subject will be briefly discussed.

The classic paper on public fire protection is that of Metcalf, Kuichling and Hawley (6), published in 1911. The method used in this paper to determine "the approximate percentage of the total cost of water works, involved by the requirements of fire protection service," meaning the percentage of interest, depreciation and taxes, exclusive of "operation and maintenance charges," was as follows:

1. Determine the percentage of the *carrying capacity* of the distribution system chargeable to fire protection on the basis of the ratio between fire demand and total demand, including fire.

2. Determine the percentage of the total cost of the water works invested in the distribution system.

3. Multiply these two percentages together to obtain the percentage of distribution system cost chargeable to fire protection.

4. Add to the product obtained under Paragraph 3 an amount, *based on judgment,* to cover "some slight additional cost in supply and reservoir systems, and substantial increase in cost of pumping capacity."

The difficulty in applying this method is that the element of judgment, covering that part of the plant other than the distribution system, is so large in relation to the "computation" on the distribution system that no precise application is possible. In a separate memorandum to the committee, the author has shown that the percentage added by "judgment" varies from 19 in a town of 10,000 population to 813 in a city of 300,000 population.

Among the other articles in the literature which present differing methods of computing fire protection, two are outstanding—one by John W. Alvord (7), published in 1914, and the second by Robert Nixon (8), published in 1937. Alvord says that:

> The proper method of dividing the cost of public and private services is [1] to make a valuation of the plant; [2] to make a study of the value of a theoretical plant, sufficient in capacity for fire service only; and [3] a study of the value of a theoretical plant, capable of domestic service only. The sum of the [2] and [3] study will give a value much greater than the value of the combined plant, and the value of the combined plant should then be divided between fire service and domestic service in proportion to the theoretical values found necessary in each case.

On the Alvord basis, it appears that a very considerable proportion, but less than 50 per cent, of the costs would be allocated to fire protection, and this rule would apply regardless of the size of the city. On the Kuichling basis, however, only the *incremental* costs are used, and the larger the city, the smaller the per capita charge against fire protection. The paper by Robert Nixon, which presents the method used by the Wisconsin Public Service Commission, also allocates the capital costs on an incremental basis.

Of the three methods mentioned above, the Alvord basis would give, by far, the largest result. The author has not noted any situation in which it appears to have been applied to its ultimate requirement. A smaller total fire charge would result from the application of the Kuichling method. Although the Kuichling end results have been frequently referred to, and no doubt considered, by those interested in computing a fire charge, one may

infer from the A.W.W.A. survey (4) that there are few if any cities where the fire charge corresponds to a *complete* application of the Kuichling method. From the meager data available, the author concludes that the Wisconsin basis generally results in a charge of 50–70 per cent of what would be obtained with the Kuichling method. The Wisconsin basis appears to have been applied consistently, so that cities of less than 50,000 population receive about 20–30 per cent of their gross revenue from the fire charge. Elsewhere in the nation, with few exceptions, the charge, where made at all, is much lower.

The differences between the methods proposed for computing fire protection costs arise in part from certain basic controversies found in the literature throughout the years. Some of these controversial matters will now be considered.

Relative Importance of Services

On the relative importance of fire protection as compared with general water service in the average community, one finds three viewpoints in the literature: [1] that fire protection is of *prime* importance; [2] that the two are of *equal* importance; and [3] that fire protection is of *less* importance than the combined needs for all other services.

On the first assumption, the cost of a system to provide fire protection alone would be compared with the cost of a system to provide both, and the *increment only* would be charged to normal uses. This assumption dates back to a time when many plants were installed mainly to furnish fire protection.

If the services are of *equal* importance, all costs of the combined system should be allocated to the two uses on the basis of the estimated costs of two hypothetical systems, one for fire and the other for general service. This is the assumption adopted by John W. Alvord.

The third viewpoint recognizes that conditions have changed since the earlier years and the municipal supply has become so vital to the health and business of the community that the general service has become the more important aspect of a water supply. Hence, fire protection should be charged only with the increment in cost required to enlarge the plant to meet fire demand.

Both the Kuichling and Wisconsin bases are premised on the general assumption that, as far as capital costs are concerned, only incremental costs are chargeable to fire protection. The differences in results arise out of differences in other assumptions.

Ratios of Demand

The second controversial matter has to do with the ratio between the amount of the fire demand and the amount of demand for all other purposes, which establishes the increment of capacity chargeable to fire. Three standards for fire demand, based on population, have been used (6): [1] those of John R. Freeman, who suggested, in 1892, both the maximum and minimum number of fire streams; [2] those proposed in 1897 by Allen Hazen and Emil Kuichling; and [3] those submitted by the National Board of Fire Underwriters in 1910. The values resulting from the several formulas which have been presented to the committee vary considerably. One example may be given here as a comparison between the Kuichling and Wisconsin bases. For a town of 50,000 population, the Kuichling basis deter-

mines the *carrying capacity* of the distribution system chargeable to fire by using a fire demand of 4,950 gpm.; for a town of the same size, the Wisconsin commission, using the *minimum* Freeman curve and assuming 200 gpm. per fire stream, adopts a fire demand of 2,700 gpm. for determining the percentage of the *cost* of the distribution system chargeable to fire.

Recently, on May 4, 1948, the National Board of Fire Underwriters issued a bulletin (9) containing "material modifications" of the old rules, which were based on total population only. In this bulletin, the requirements for residential areas and industrial plants are separately discussed.

The determination of the second factor in this ratio, the amount of *demand for all uses other than fire* (which may occur during a fire), also offers opportunity for a difference of opinion. In the Kuichling standard (6), this maximum rate was assumed "to be *twice* the average daily rate therefor, in the plant furnishing only domestic service, and 1½ times the average daily rate therefor in the plant furnishing both domestic and fire protection service."

The underwriters state that "a system must be capable of supplying a reasonable quantity for fire protection for a period of ten hours, in addition to domestic consumption at the maximum daily rate, for any day within the past three years." The Public Service Commission of Wisconsin, however, has based its computations on the maximum *hourly* rate. Obviously, differences in assumptions on this one item result in marked differences in results.

Operation and Maintenance Costs

The foregoing relates to capital costs. A review of the literature also shows a wide divergence of opinion on the amount of operating labor and maintenance chargeable to fire protection.

If fire protection is of *equal* importance with normal service, the ratio of the operating costs of each of the two hypothetical systems will determine the percentages of the operating costs of the combined plant chargeable to each service. This is the logic of John W. Alvord. He assumed that the plant built for fire protection would not be in operation 99 per cent of the time, but that the engines would be warmed up, the pumps turned over once in a while and everything, including the operating force, would be ready for a fire. On such a basis, more than 50 per cent of the operating expense of the combined plant would be chargeable to fire.

But, on the other hand, if one follows the logic of the *incremental* theory, the domestic plant would carry its full quota of operating labor, and only the *additional* labor and expense required in a larger plant would be chargeable to fire. Generally, this charge would be relatively a small amount.

Kuichling (6) said, in reference to operating and maintenance costs, that the "data are meager [but] it is believed that these charges lie between 5 and 10 per cent of the total annual operation and maintenance charges, including taxes, but excluding depreciation, interest and profit allowances." In *Water Works Practice* (10), data are presented on the "annual operating and maintenance costs attributable to fire protection service, Indianapolis Water Company," as submitted in 1924 to the Public Service Commission of Indiana. In this tabulation, approximately 25 per cent of the pumping expenses and nearly 40 per

cent of the distribution expenses, plus other items, were allocated to fire protection.

The Wisconsin Public Service Commission allocates operating and maintenance expenses under a set of rules which first divide expense into "capacity" and "output" costs and then assign "capacity" costs to general service and fire protection on the basis of the ratio of fire demand to total demand, while "output" costs are determined on the basis of water actually used for fire. These rules are fairly complicated and no two engineers, working independently, would be likely to arrive at the same total. In the example given by Nixon (8), more than 20 per cent of the total annual expense is charged to fire protection, which would appear to be in excess of the amount that would result from the strict application of the incremental theory, based on a comparison of two hypothetical plants.

Basic Assumptions

From the foregoing discussion and illustrations, and from many other examples in the literature, it is apparent that the subject of fire protection is in great confusion as to both the theories involved and the methods of application. This confusion goes back through the years and the thinking of many writers and the decisions of courts and commissions.

In spite of this background, however, the committee has the temerity to set as its objective the outlining of a basis for the computation of fire protection charges that will be relatively simple in its application and yet reasonable in amount in the light of present conditions.

The following basic assumptions are presented for consideration:

● 1. The prime function of a water works today is to provide general service to water users, and only the *increment* in costs, both capital and operating, is chargeable to fire protection.

● 2. The demand of water for fire—both the amount and the duration in hours—may be based, for the present, on the standard requirements of the National Board of Fire Underwriters.

● 3. The percentage of *plant capacity* required for fire may be taken as the ratio of the "fire demand" to "fire plus general service demand" for the *duration in hours,* as specified by the underwriters; and the percentage of *plant cost* chargeable to fire protection would be the percentage of capacity multiplied by the ratio of increased cost to increased capacity.

● 4. The dollars chargeable to fire protection, on account of capital costs, would be obtained by multiplying the percentage of *plant cost* obtained under Paragraph 3 by the interest, taxes, depreciation and return on investment for the *applicable* portions of the plant; plus all charges involving hydrants and their connections.

● The applicable portions of the plant would include all production and distribution facilities but would exclude hydrants and connections, consumer's services and meters, and most of the investment in office and shop.

● 5. The dollars chargeable to fire out of operating and maintenance expense would be based on an estimate of the *additional expense only* required by a larger system, on the assumption that the system needed to supply general service only would require a complete labor force, office, shop and administrative overhead; plus the cost, at the existing commodity rates, of the water actually used for fire.

● 6. If storage is available *in excess of* the needs for general service, all costs connected with such storage should be charged directly to fire; but such excess storage will reduce the fire demand on the balance of the system and hence reduce the ratio of fire demand to total demand and the percentage of the balance of the system chargeable to fire. Excess raw-water storage will reduce the ratio applicable to the supply facilities; excess clear-well storage will reduce the ratio applicable to production facilities, except high-lift pumping; and excess elevated storage will reduce the ratio applicable to high-lift pumping, transmission and distribution, except in that portion of the distribution system between the elevated tanks and hydrants.

These bases for the determination of the fire protection charge are submitted to the committee and the Association for their constructive comment.

Rate Structure

The next subject to be discussed is the distribution among water users of the balance of the revenue required by the water system—that is, the commodity rate structure. The items to be considered are, first, the service charge, whether based on customer costs or on "demand" or "ready-to-serve" costs, or both; and, if the demand costs are to be included, the basis for fixing their amount and the basis for their distribution among customers. Second, rules are desirable for the determination of the number of steps, the unit rates for each step and quantity limitations within which each unit rate shall apply.

Service Charge

Although a "service charge" has long been recommended as a proper feature of a rate schedule, it has not been generally adopted, as such, in rate structures. Apparently it has not been understood by the public and has, therefore, been rejected, although minimum charges have been accepted and minimum bills, in combination with adjusted unit commodity rates, are used to produce the equivalent of a service charge. These matters are details that can be worked out. The real problem is to determine the equitable allocation of costs between users of different classes; and this determination involves the service charge.

The Committee on Water Rates of the Michigan Section, A.W.W.A., gave careful consideration to the "demand" charge and discarded it as a basis for the service charge to the great majority of water users. It is a theory applicable to fixing fire protection charges, and it may be applied equitably to large users served through recording meters which supply the necessary evidence of "demand." The development of a maximum-demand water meter would extend the field of application of the demand theory to correspond with its applicability in the electrical industry, from which the theory was derived.

At this point, it may be advisable to mention a difference between "ready-to-serve" and "demand" costs that could well be accepted, although to date these terms have been used interchangeably. A "ready-to-serve" charge could be applied when a water main is in the street in front of a vacant lot, and there is much justification for a

charge under such circumstances. A "demand" charge, on the other hand, should not apply except to *users of water,* either intermittent or regular. Such a charge would presumably be based on the ratio of the maximum rate of demand to the average rate of use.

In rejecting the demand theory, particularly as applicable to the domestic user of water, the Michigan committee returned to the basis originally adopted by both the New England Water Works Assn. and the A.W.W.A.

In 1914 and 1916 an N.E.W.W.A. committee on water rates recommended a service charge covering: [1] all costs connected with service pipes and meters; [2] the cost of reading meters, keeping records, billing and collecting; and [3] the cost of all "lost" water, "reckoned at the lowest rate charged for any water that is sold" (11). This report was the subject of much discussion, the opposition contending that the service charge should also include "those elements of cost which are proportionate to the maximum demand." This argument was premised on a "method set out by Hopkinson of England, and Henry L. Doherty in this country, treating of rates for electrical supply."

According to the latter theory, the service charge should include all *customer* costs, "such as billing, meter reading, accounting, collecting, meter testing, etc.," plus all *demand* costs—including mainly profit, taxes, insurance and depreciation—not chargeable to fire protection. The balance of the expense, chiefly pumping costs, determined the commodity rate. In an example given, about 85 per cent of the total expense, exclusive of the fire charge, was allocated to the service charge, and the balance, or 15 per cent, determined the commodity rate.

In 1917 another writer (12) took vigorous exception to the recommendations of the N.E.W.W.A. report, insisting that the service charge "should be constituted of a consumer charge and a capacity charge"; the capacity charge should "be the amount necessary to keep the plant ready for service and intact were not a drop of water delivered for the entire year" (including interest and depreciation on the entire plant); but "maintenance and profits should be incorporated in the output charge." This writer's position was that "the function of a service charge is more to stabilize than to produce a revenue" and, if "due to the ups and downs of prosperity," too much dependence is put on the output charge, "the revenue will at times become inadequate." But he also aimed at a commodity rate that would enable a water works to justify sales to "a large consumer . . . at any rate which . . . will add to the net revenue of the plant, even though the rate is below the average cost as built up by administration, operating, depreciation, interest and profit." One may conclude that no such basis of justification for an excessive service charge should be considered by the committee.

The answer of Allen Hazen (13) to the argument for this basis was that, if it "is carried out to its logical conclusion, it puts too large a burden on the smallest takers"; and "that the method of calculation" of the N.E.W.W.A. committee "puts all the load on the smallest consumers . . . that is their fair share."

In 1923 the A.W.W.A. accepted the report of its committee on a standard form of rate schedule, which was in

accord with the previous recommendations of the N.E.W.W.A. committee. *Water Works Practice* (10), however, neglected the recommendations of the 1923 committee and reverted to a basis for the service charge substantially as proposed by previous advocates of a relatively high charge, except for the following qualification: "It sometimes happens that an analysis of cost in a particular system carried out in this way will result in putting a considerable proportion of the total cost on the service charges. *Theory should not be carried to extremes,* and regardless of calculation, it is best to limit the service charges in all cases to reasonable amounts, not greatly exceeding those actually adopted in other cases. *The consumption charges should, in all cases, carry the major part of the entire cost of service.*"

Down through the years, the advice of the manual appears to have influenced the thinking of many writers, each making his own interpretation, with an end result of confusion and a wide diversity in schedules. In fact, only a few schedules appear to have followed the "demand" theory to extremes, and, although the writers claim to have used the theory, the service charges proposed by them are not materially higher than a proper customer charge would be.

The reasons for rejecting the "demand" charge as an element of the service charge were set forth at some length in the Michigan Section committee report (14) published in 1949. The details need not be repeated, but, briefly, there are three reasons:

1. There appears to be no logical place along the line at which one may stop in determining the amount of demand charge to be included. The theory cannot be applied except arbitrarily or on the basis of judgment and precedent elsewhere.

2. There is no basis for the equitable distribution of the demand charge among water users, except through the development of maximum-demand meters or for large users served through recording meters. The past practice of distributing a demand charge on the basis of meter capacity ratios is not equitable in that it ignores all differences in conditions among 90 per cent, more or less, of all customers and assesses a considerable part of the whole cost uniformly upon these customers.

3. The addition of a demand charge to a proper customer charge will invariably place an undue burden on the smaller user and result in too low a commodity rate to the larger user. An application of the effect of such a combination on the rate structure of Detroit has been made by Hal F. Smith (*see* p. 1000, *this issue*).

The present author, therefore, recommends that a service charge include:

● 1. All commercial expense and a suitable portion of the administrative expense and overhead costs, the total of which is to be divided *equally* among all accounts.

● 2. All expense arising out of meters and services, with a share of administrative expense and overhead, the total of which is to be distributed among accounts on the basis of *meter costs.*

● There should also be a minimum charge, based on the commodity rates. For the smallest meter, this minimum should be the service charge, plus the cost of, say, 100 gpd. per meter. And it has been suggested that a minimum charge for all other sizes of meters be made on the basis of the service charge for each meter, plus the cost of a mini-

mum quantity of water allowed for each meter, based on the squares of the diameters of the several meters.

Commodity Rates

Considering the commodity rates, there are three questions to be answered: [1] how many steps are desirable? [2] how are the unit rates determined for each step? and [3] what are the rules for fixing step quantity limitations? The following answers to these questions are presented in the Michigan Section report (14):

● 1. In line with the recommendations of *Water Works Practice*, three steps are suggested, referred to as domestic, intermediate and wholesale.

● 2. Unit rates are determined thus:

● *a*. The *wholesale* unit rate is obtained by dividing the total of all production and transmission costs (capital, operating and maintenance)—excluding deductions chargeable to fire protection and including a portion of administrative expense and overhead—by the total annual pumpage.

● *b*. The *intermediate* unit rate is obtained by adding the intermediate increment to the wholesale rate. The intermediate increment is obtained by dividing the total costs—exclusive of deductions chargeable to fire protection—arising out of distribution mains larger than 6 in. by the water sold through all meters, except sales at the wholesale rate.

● *c*. The *domestic* unit rate is obtained by adding the domestic increment to the intermediate rate. The domestic increment is obtained by dividing the total costs—exclusive of deductions chargeable to fire protection—arising out of distribution mains 6 in. and smaller, plus the cost of all lost water, by the water sold through all meters ¾ in. and smaller.

● 3. Step quantity limitations must be adopted which, when applied to the unit rates, will produce the necessary revenue. As an initial determination, it is suggested that the domestic rate apply up to a quantity equal to twice the average rate of use through all domestic meters, ¾ in. and ⅝ in.; that the intermediate rate apply up to a quantity equal to twice the average rate of use through all intermediate meters—say, 1, 1½ and 2 in.; and that all quantities over the intermediate limit fall under the wholesale rate.

The foregoing rate structure is offered as a basis for the average city and the average customer using year-round service in a normal manner. It does not provide the answer for those special situations involving intermittent and abnormal uses of water. On some of these problems, the committee hopes to make specific comments later.

General and widespread comment on all of the above, from all interested parties, is earnestly solicited by the committee.

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Discussion

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Fixing a universal standard for the amount of revenue that a water works should receive is out of the question, as there are too many local factors involved in the operation of a water system. The writer believes, however, that the A.W.W.A. rates committee can formulate or outline the fundamentals of a schedule to be followed by systems operating under different conditions—for instance, those with well supplies rather than surface sources.

Inasmuch as more than 85 per cent of the nation's plants are publicly owned, the committee's work should be confined to such systems. Privately operated plants should, however, be studied in this connection for comparison purposes. Nevertheless, there seems to be no reason why the approach to the subject of revenue should not be identical for both privately and publicly operated plants: both are striving to do a certain job—that of supplying water to the citizens of a municipality. The method of disposition of the revenue is the only essential difference, the privately operated plant requiring sufficient revenue over and above its total expenses to pay dividends to its stockholders, while the publicly operated plant has no dividends to meet.

A privately operated utility is entitled to a fair and just return on investment. In the writer's opinion, 5–8 per cent, based on active investment, is sufficient. Similarly, a publicly operated utility is entitled to a return, at least on the current active investment, enabling it to meet all obligations such as taxes (including all taxes now paid to cities by privately operated utilities) and payments to the city for actual services received and property utilized. On the other hand, the city should pay the water department for all services rendered to municipal departments, so that all the moneys received and water furnished by the utility can be properly accounted for.

To tell how much revenue is necessary to operate a water works plant, it is agreed by all that a proper accounting of operating and maintenance expense, depreciation (a subject in itself), taxes and capital costs is required. Such information is needed to arrive at a judicious rate schedule, whether the utility is privately or publicly operated.

The existence of a municipal public water system is primarily for the benefit of the citizens, and its cost should be borne by them in proportion to their liability, with no discrimination. In the writer's opinion, the nontaxpaying water user cannot be placed in the same category as the taxpaying water user, because the latter, in owning property, takes on added responsibility to the community. If water works are financed through tax bonds, the taxpaying user pays doubly for the same service or privileges as the nontaxpaying water user receives. Even if the taxpaying user does not have to pay taxes on a water bond issue, he is still a more substantial citizen because of the fact of being a property owner. Should any favor in water rates or service be granted to any class of customer, it should go to the taxpayer. The writer is, however, against the adoption of a rate favoring any class of customer except on a volume basis applied to all users alike. Experience has shown that discrimination will cause trouble. But, if a surplus of revenue is found, it should by some means be placed to the credit of the taxpaying water user and not be diverted to other municipal departments.

Fire Protection

The allocation of revenue between taxpaying and nontaxpaying water users brings up the question of the percentage chargeable to fire protection. In this regard, the taxpaying water user, because of the protection given his property—automatically reducing his insurance rate—should rightfully be charged with the bulk of the fire protection obligation. Since the nontaxpaying user does not own real property, the protection he receives extends chiefly to household goods, representing a very small insurance saving; therefore, his share of the fire protection obligation may be paid in the general service charge.

In the writer's opinion, fire protection service is neither of prime importance nor of equal importance with general service, considering the role played by a pure water supply in improving public health and increasing property values.

A water system which is capable of maintaining a firm domestic pressure of 50 psi. at a high elevation in any high-value residential section and does not suffer more than a 10-psi. reduction during a peak summer day's consumption will afford sufficient fire protection. The writer believes that *adequate* domestic pressure practically insures fire protection, because, to maintain such pressure, it takes a well gridironed system of 6- and 8-in. mains, with 10- and 12-in. reinforcement at half-mile intervals. Inasmuch as the general service represents more than 90 per cent of the annual pumpage, it requires the bulk of the physical equipment of the water system.

Fire hydrants should be renamed, as they are used more for general service than for fire protection. In fact, the general service could not be operated without hydrants or equivalent openings about 1,000 ft. apart in the system. In their absence, the flushing and repair of mains, the sprinkling and washing of streets, the cleaning of storm sewers and the like could not be effectively accomplished. The use of the hydrant in general service as well as in fire protection meets the requirements of both services at a minimum cost. Although this practice may not receive the approval of the fire protection experts, it has actually been found satisfactory for a progressive, rapidly

growing city of more than 160,000 population. It should be said that these observations may not apply to cities of over 500,000 population. Such cities should be the subject of individual studies, but they constitute only a small percentage of the municipally operated plants.

A reasonable estimate of the amount chargeable to fire protection is 10 per cent. In the writer's own experience, it has never been necessary to raise the pressure to meet fire demands. Consequently, it is needless to worry too much about fire protection. The A.W.W.A. should not adopt somebody else's figures for fire protection but should work out its own plan, based on information secured from the municipally operated systems and keeping in mind the taxpayer who owns the utility and pays the insurance.

Some beautiful equations have been formulated to arrive at the proportion of the cost of the system chargeable to fire protection, but, in actual operation, the water to fight the fire must be delivered in time and in sufficient quantity. If a valve failure prevents this, it makes little difference how the costs were prorated. In fixing the fire protection charge, therefore, it must be admitted that the element of judgment plays an important part.

Following the logic of the incremental theory, the domestic or general-service plant would carry the full quota of labor and only the additional labor and expense—a relatively small amount—required in a larger plant would be chargeable to fire. This is in line with the writer's view that the general service has become of greater importance, particularly in the last ten years, which have brought new uses for water in household activities. The writer is in agreement with the committee objective of outlining a basis for the computation of fire protection charges that will be relatively simple in its application and yet reasonable in amount in the light of present conditions.

Rate Structure

The rate structure is, without a doubt, the most important item in the operation of a water department. On it depends the efficient functioning of the system. So much difference of opinion on this subject exists in the minds of eminent engineers, ratemakers and even the courts that it seems advisable to start from scratch. Where there is so great a difference of opinion, there is error. The writer is at a loss to understand why, despite the time and money spent on the problem, there is still confusion, unless it is due to the fact that both public and private ownership are involved, complicating the issues.

Possibly an attempt is being made to be too scientific in the matter. Allocating portions of the total cost to specific parts of the physical plant or service ignores the fact that, in the operation of the system as a whole, the parts are interdependent. Therefore, it seems advisable to base revenue requirements on the entire cost of the system—including the value of the physical plant, the expense of all phases of operation and the cost of current improvements necessary to carry on its activities—and make a rate accordingly.

To get closer to the subject, the commodity rate classifications suggested—"domestic," "intermediate" and "wholesale"—are good, but the terms "domestic," "commercial" and "industrial" might be more appropriate for identification of the type of customer. For instance, "domestic" suggests household usages, ordinarily measured by ⅝–1-in. meters. "Commercial," a

step higher than domestic, connotes such users as apartment buildings, hotels, restaurants or business concerns—served through $\frac{5}{8}$–4-in. meters. The third classification, "industrial," speaks for itself. To the writer's mind "wholesale" is too broad a term for this class.

The writer feels that the service charge is the fairest and best method of making a flat charge. At Shreveport, however, a minimum charge is used, with little or no complaint. It is believed that the customer would be willing to pay a flat service charge plus a commodity rate charge for actual water used, especially if the rate remains within reason. The writer is open to both the service and minimum charge plans, but not to the use of a demand or ready-to-serve charge.

The service charge should be set up to recover the full cost of the meter installation over a 20-year period. The basis for the service charge should not include any portion of the cost of billing, meter reading, accounting or collecting, but should cover the cost of cleaning and testing the meter for correct registration every six to eight years.

Except for a small charge for fire protection and the service charge, the commodity rate should carry the costs of the system. In the writer's estimation, the simpler the rate structure, the better. The more easily the customers understand it, the better they will feel about it and the less difficulty there will be in working it out. Primarily, the business of a water utility is to sell water and obtain its revenue from that source. Therefore, the rate should be based on the cost of the commodity produced and sold. The service charge is only incidental, and, based on the items referred to above, 25¢ per month for a $\frac{5}{8}$-in. meter will be sufficient under ordinary circumstances.

The service charge is fair and equitable because it is optional with the customer to use a $\frac{5}{8}$-, $\frac{3}{4}$- or 1-in. meter on his supply, paying accordingly. For the water itself, he pays under the same schedule of rates as his neighbor with a 2-in. meter. Thus, the small customer saves on the service charge. The demand or ready-to-serve charge, unlike the service or minimum charge, usually places the burden on the small customer.

One reason for lightening the load on the small customer is that the commercial-intermediate and industrial-wholesale users, being business concerns, are better able to absorb any increase in rates or can raise the price of their product to include it. At Shreveport, the domestic users account for 26.4 per cent of the consumption and pay 48.2 per cent of the revenue, while the commercial or intermediate users account for 36 per cent of the consumption and 27.7 per cent of the revenue, and the industrial or wholesale users, for 37.6 per cent of the consumption and 24.1 per cent of the revenue. This situation should be corrected in favor of the domestic customer, who is usually an individual taxpayer.

The writer does not believe in loading the service charge with operating costs, regardless of the type involved. No item of operation can be left out and still permit the delivery of pure, wholesome water to the meter. The full cost of the water includes the cost of purification, maintenance, current improvements and so forth. Consequently, the total expense must serve as the base for the rate schedule.

Steps in Schedule

Having arrived at the cost of delivering the water to the customers' property, it should be apportioned in as few steps as possible. The Shreveport rate schedule has five steps and is very handy, but a smaller number would be preferable. There is just not enough difference between the customers to require splitting up the schedule into more than five steps.

The first step ("domestic") can be 20,000 gal. per month with the next 100,000 gal. constituting the "commercial" or "intermediate" step. The next 300,000 can serve as a balance quantity by being placed between the second step and the "industrial" or "wholesale" step. The industrial-wholesale rate would then apply to quantities over 500,000 gal.

Applying these principles to Shreveport, the following rates result:

Quantity per Month *gal.*	Rate per 1,000 gal.	Price
First 20,000	$0.30	$ 6.00
Next 180,000	0.25	47.00
Next 300,000	0.14	42.00
Over 500,000	0.12	60.00
	Price per mil.gal.	$155.00

The price of $155 per million gallons is 43.2 per cent higher than at the present Shreveport rate, which was adopted in 1917.

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There is much sound judgment and justification for the conclusions reached by Chairman Ayres as a result of his studies. Some comments in the way of constructive suggestions are offered by the writer.

Debt Requirement

It is common practice to meet actual interest and bond retirements out of revenues. The difficulty is that, in a community having little growth, the present generation is paying for the benefit of future users. Of course, this fact should eventually be reflected in a low overall interest rate, as indicated by Ayres. In spite of the variations in the actual names given to bonds, there is little difference in the ultimate result whether the title is general-obligation, special-assessment, mortgage or revenue bonds. As suggested in Ayres' report, it is customary for all such bond retirements to be considered as payable out of revenue, and it is usually essential that the custom be continued.

Reserves

Where extensions are of considerable magnitude, they could be temporarily financed out of reserves and could be bonded at intervals such as every five years, in conjunction with the periodic review of funds suggested by Ayres. It may be argued, however, that such a procedure is not feasible and there may be some state or local laws that would prevent it.

The difficulty is to prescribe a method by which the amount or percentage of extension costs to be financed by periodic bond issues should be determined. As a suggestion, assuming, for example, a depreciation of 1 per cent on the distribution system, all main extensions exceeding this figure might be bonded. The reasoning is that depreciation charges already cover the money needed to keep the investment intact and expenses above that amount may be legit-

imately charged to future customers. The one fallacy in this method is that the dollar value fluctuates and replacements or additions are not always comparable in cost.

Taxes

While the publicly owned utility should pay taxes, payment by the municipality for fire protection and other service to the municipality should be near the same figure. A review of many of the private utilities in New Jersey shows that local taxes and repayments for fire service to the utilities by the municipalities are somewhat equalized. The main fault with this situation is that the cost of fire protection is usually well above the approximately 12–15 per cent of gross revenue received for that service. Until a more equitable figure for fire protection is established, the tax levy should probably not exceed 10 per cent of gross revenue.

Contributions

It is realistic to recognize that most contributions to public budgets out of water revenues, when once started, cannot readily be discontinued. The suggestion is made that the governing body be persuaded to permit a reasonable increase in rates, giving part of the additional revenue to the municipality and using the balance for those extra improvements to the water works system that it often needs and never gets. Certainly, no one can argue that water rates have, in general, kept pace with the increase in cost in most other public services.

Ratios of Demand

It may be of interest to note that in 1949 two residential communities in New Jersey had peak-rate usages in extremely hot weather representing a maximum hourly ratio of approximately four to one over the annual average. The largest city, Newark, had a peak ratio of two to one.

Basic Assumptions

In allocating *increment* costs to fire protection, it may be well to add that air conditioning, lawn sprinkling and other such peak-time uses should have an increment cost added to their normal costs.

In some instances, it seems probable that additional distribution storage would be cheaper and more dependable than increased or forced pumping or main capacity.

The 5–10 per cent for fire charges advocated by Kuichling may be too low for present-day conditions. No figures are available at this moment, but indications are that the matter is being or will be studied and that a reasonable answer will soon be forthcoming.

Service Charge

The writer has frequently expressed definite opposition to the service charge and still favors a minimum charge. Two portions of Ayres' discussion on this subject deserve special commendation. One is the plea for a maximum-demand water meter. The other is the explanation of the true meaning of "ready-to-serve" and "demand" costs.

The minimum charge, permitting 100 gpd. per meter or 9,000 gal. per quarter, seems approximately to fit the conditions usually advocated by water works men.

Hal F. Smith

Sr. Admin. Asst., Dept. of Water Supply, Detroit.

Included in the discussion (1) presented by the A.W.W.A. rate committee in 1949 was an application of the water rate formula proposed by the Michigan

Section rate committee (2) to the costs and related data of the Detroit Dept. of Water Supply. The purpose of this application was to illustrate the kind of rate structure which the proposed formula would produce.

TABLE 1

Application of Rate Formulas to Detroit (1947–48 Costs—City Users Only)

Consumption Charge		
Monthly Use *cu.ft.*	Rate *A*	Rate *B*
	$/1,000 cu.ft.	
First 3,000	0.83	0.18
Next 200,000	0.55	0.12
Over 203,000	0.46	0.10
Monthly Service Charge		
Meter Size *in.*	Rate *A*	Rate *B*
	$	
$\frac{5}{8}$	0.35	1.39
$\frac{3}{4}$	0.43	1.99
1	0.54	2.89
$1\frac{1}{2}$	0.81	4.99
2	1.12	7.39
3	1.73	12.18
4	3.26	24.18
6	5.56	42.16
8	7.86	60.15
10	10.93	84.13
12	13.23	102.12
Per cent of total revenue produced by service charge	18	82

In the discussions which followed the presentation of the committee report, there appeared to be no objection to this rate structure, but there was some disapproval of the fact that the formula did not include ready-to-serve costs in the service charge. Obviously, if such costs are included in the service charge, the resulting rate structure will be vastly different. In order to show exactly what this difference would be, Table 1 compares the rate structure based on the application of the proposed formula to Detroit data (Rate *A*) with the rate structure that would result from applying the same formula to the same basic data, but with ready-to-serve costs transferred from the consumption portion of the rate to the service charge (Rate *B*). Table 2 shows the result of applying both of these rate schedules to given consumption and meter sizes, together with comparative related data.

In this discussion, the term "ready-to-serve costs" is used as defined by *Water Works Practice* (3):

> . . . to include all costs of operation involved in keeping the plant in readiness to serve all consumers but not actually supplying them with water. The interest, taxes, depreciation and a certain portion of operating expenses must be met, regardless of whether the consumer takes any water or not. These expenses are known as "ready-to-serve costs," and it has been deemed proper that this class of service should be covered by a rate independent of the charge for water actually consumed. Therefore a service charge is made to cover the above-mentioned service.

This definition, but not necessarily the theory, appears to have been generally accepted by the industry. It does not, however, specify what portion of operating expenses should be included.

The most complete allocation of operating expenses which could be found in the literature was that made by Robert Nixon (4). This is the allocation used, as nearly as possible, in applying the formula for Rate *B* to Detroit data.

Regardless of what one may think of the theory of including ready-to-serve costs in the service charge, it is con-

TABLE 2

Comparison of Rates A and B for Various Consumptions and Meter Sizes

Meter Size *in.*	Quarterly Use *cu.ft.*	Quarterly Charge—$		Avg. Rate—*$/1,000 cu.ft.*		Ratio of Avg. Rates to Lowest Avg. Rate	
		Rate A	*Rate B*	*Rate A*	*Rate B*	*Rate A*	*Rate B*
5/8	1,000	1.88	4.35	1.88	4.35	4.0	33.5
5/8	2,500	3.13	4.62	1.25	1.85	2.7	14.2
3/4	5,000	5.44	6.87	1.09	1.37	2.3	10.5
1	10,000	9.64	10.41	0.96	1.04	2.0	8.0
2	100,000	60.88	34.71	0.61	0.35	1.3	2.7
4	500,000	287.30	133.08	0.58	0.27	1.2	2.1
6	1,000,000	534.01	239.20	0.53	0.24	1.1	1.8
12	10,000,000	4,697.02	1,319.08	0.47	0.13	1.0	1.0

tended that such inclusion produces a rate which would not be acceptable to the customers and would not be adopted by water departments. Furthermore, the ready-to-serve costs are either to be included in the service charge or to be excluded, and there is no logical basis for a compromise between these two extremes, as is suggested by *Water Works Practice* (3):

> It sometimes happens that an analysis of cost in a particular system carried out in this way will result in putting a considerable proportion of the total cost on the service charges. Theory should not be carried to extremes, and regardless of calculation, it is best to limit the service charges in all cases to reasonable amounts, not greatly exceeding those actually adopted in other cases. The consumption charges should, in all cases, carry the major part of the entire cost of service.

These are the reasons why the water rate formula proposed by the Michigan Section rate committee does not include the ready-to-serve costs in the service charge.

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Distance and Demand Factors in Suburban Water Rates

By Melvin P. Hatcher

A paper presented on May 25, 1950, at the Annual Conference, Philadelphia, by Melvin P. Hatcher, Director, Water Dept., Kansas City, Mo.

THE Kansas City, Mo., municipally owned water works system provides water for service to about 575,000 people. Of this number, about 440,000 are residents of Kansas City proper and are served directly from city-owned mains. With negligible exceptions, the city renders no direct service outside its borders. The 135,000 people living in areas outside of and adjacent to Kansas City who obtain water from the city system are served through distribution networks owned and operated by private water utilities, public water supply districts, real estate companies and others. Kansas City's rates for service to these wholesale purchasers are the subject of this article.

Suburban Sales

Kansas City is bounded on nearly all sides by areas using water from its system. These areas are located in two counties in Missouri and one in Kansas. The point of connection between the city's and the purchaser's mains is usually at or near the city limits. There are 28 metering stations serving the agencies which render water service outside the city, each having a maximum daily demand of as much as 10,000 cu.ft. or 75,000 gpd. Figure 1 shows the arrangement of these points of purchase (indicated by black dots) around the perimeter of Kansas City. The total revenue from water sold for use outside the city is about $360,000 per year, the average rate being approximately 10.7¢ per 100 cu.ft.

From 1940 to 1948 Kansas City dealt with rates for water sold for use outside its borders in the same way as many other cities: it devised a rate for such service that was somewhat higher, uniformly, than the city rate. The principle involved in this practice is that water users outside a corporate area do not share in the responsibilities of ownership of the supply facilities and are therefore not entitled to its benefits.

Prior to 1940 water for suburban consumption had been sold at city rates. One water company, which had been purchasing more than half of all the water sold for such use, had a 25-year contract (due to expire in 1948) at city rates. On this basis, the company's costs for water had been averaging about 6.25¢ per 100 cu.ft. In 1940, however, the charges for all other suburban use were raised to a level about one-third higher than city rates and netted a return averaging about 9.25¢ per 100 cu.ft.

The latter rate plan was adhered to until service was extended to the city of Lee's Summit, Mo., in 1946. Service to this city of 3,000 persons required

a connection at the far southeast corner of Kansas City's distribution system, which would entail more than the average amount of transportation expense so that the prevailing rate would not produce an adequate return. Therefore, the contract for this service was negotiated on the basis of a special rate involving a commodity charge about 13 per cent higher than prevailing sub-

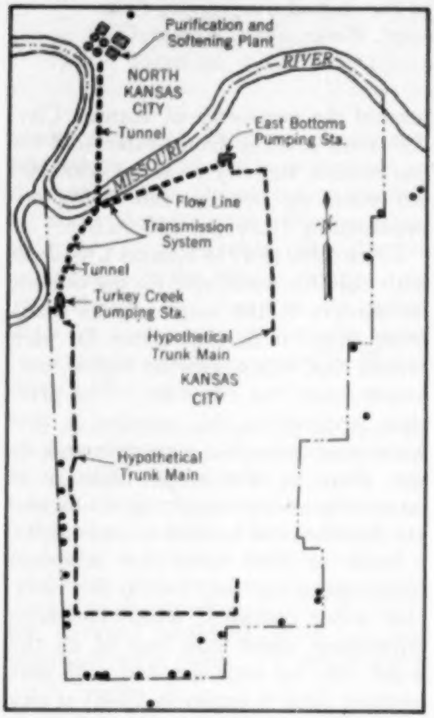

FIG. 1. Suburban Points of Purchase

urban rates and a fixed charge for participation in the use of a particular connecting main. The revenue under this rate averaged about 12.90¢ per 100 cu.ft.

Distance-Demand Plan

Following the expiration of the special contract mentioned above, all suburban service, including that to Lee's Summit, was brought under a distance-demand rate plan on October 1, 1948. This rate has three additive parts: [1] a monthly charge which is weighted with regard to the distance from the main pumping stations and is proportional to the maximum day's demand (except Sundays and holidays) during June to September, inclusive; [2] a monthly charge which is similarly weighted and is proportional to the excess of the maximum hour's demand (between 4:30 and 8:30 P.M. on a weekday in the months named) over the maximum day's demand; and [3] a

TABLE 1

Illustration of Rate Plan

Charge	Quantity *cu.ft.*	Rate *$/100,000 cu.ft.*	Cost $
Maximum daily demand	46,000	800	368
Excess hourly demand	24,000	500	120
		$/100 cu.ft.	
Consumption	460,000	0.055	253
	620,000	0.045	279
		Total	$1,020

charge based on the quantity of water used—5.5¢ per 100 cu.ft. until a total of ten times the maximum daily demand is purchased and 4.5¢ per 100 cu.ft. thereafter.

The cost of water under this rate plan therefore varies with the point of purchase. The rate ordinance names each point of purchase and fixes the monthly charge per 100,000 cu.ft. of maximum daily demand and of excess hourly rate of demand. The rate includes the provision that demands are additive for any one purchaser having more than one point of purchase and that demands once established shall pre-

vail for the succeeding eleven months unless increased in that period. The charges per month per 100,000 cu.ft. of maximum daily demand and excess hourly rate of demand range, respectively, from about $400 to $1,200 and from about $100 to $1,000.

The mechanics of the rate plan may be illustrated by this example: Assume a point of purchase having a maximum-daily-demand charge of $800 a month and an excess-hourly-rate-of-demand charge of $500 a month; a maximum daily demand of 46,000 cu.ft.; a maximum hourly rate of 70,000 cu.ft. per day; and a consumption of 1,080,000 cu.ft. in a month. The cost would be computed as shown in Table 1.

Theory of Plan

There were quite a few reasons for adopting this rate plan, some of them possibly peculiar to Kansas City. It had become apparent at the time of the study of rates for Lee's Summit that the water sold for suburban use was not producing its proper share of revenue. This study had called attention to the importance of the role played by the transportation part of the total cost of serving these suburban consumers. Later studies revealed that the fixed charges on the trunk mains in the distribution system accounted for 25 per cent of this total cost. It appeared, too, that a straight across-the-board distribution of all costs applicable to suburban service would tend to increase the charges to purchasers near the river to such a point that it might tip the scales in favor of the development of separate supplies. Furthermore, as supplies in the Kansas City area are available only from the Missouri or Kaw River or from wells in the valleys of these streams, water used to the south of Kansas City was certain to involve a transportation expense. These conditions pointed to the need for a rate plan which would take into account the distance from the river or, more particularly, from the city's pumping stations.

This idea, in turn, fitted well with that of charges based on maximum daily and hourly demands. Variations in the use of water cause the ratio of maximum to average demand (inverse load factor) for the various points of purchase from the Kansas City system to vary over a rather wide range. The west side of the system is predominantly a high-class residential area with a high percentage of water use for lawn sprinkling. There the maximum day will run as high as 250 per cent of the average. The east side, tempered with more commercial consumption, will have maximum-to-average-day ratios on the order of 140–150 per cent. Because so much of the total cost of service to the suburban customers is a function of the maximum daily demand, it seemed desirable that the rate be made to include a charge which would be proportional to this demand.

The maximum-to-average ratios for peak hours could be as much as twice the peak-day ratios just mentioned on systems having no storage. The peak hourly demand in suburban areas, generally due to lawn sprinkling, comes late in the afternoon and is likely to coincide with the system peak hours. Although many of the suburban systems had provided storage before the distance-demand rates were initiated, some of the larger consumers in the residential areas on the west side had not, and it seemed desirable that the rate include a provision for a charge proportional to the maximum hourly demand.

The author knows of no other instance of the use of maximum demands in water rate practice, but the reader will recognize that the principles involved—except those related to distance

—have long been in use in electric power rates. The application of these principles to water rates is not particularly complex. Generally speaking, the fixed charges on the facilities devoted to the service of suburban customers are distributed with the demand charges, and the production and pumping expenses are distributed with the commodity charges. Ground level reservoirs at the Turkey Creek and East Bottoms pumping stations (*see* Fig. 1) equalize variations in hourly demands on these stations; therefore, the fixed charges on the supply, purification and pumping plants in North Kansas City are distributed with the maximum-daily-demand charges. The two pumping stations named and the trunk mains must have the capacity to meet maximum hourly demands; therefore, the fixed charges on these parts of the property are distributed with the maximum-hourly-demand charges.

The more difficult part of the fabrication of the rates is centered in the fixed charges on the trunk mains in the distribution system. A Hardy Cross analysis of the distribution system, carried out in 1944 to determine the need for improvements, was made the basis for the apportionment of these charges. An adaptation of the analysis was undertaken to show the sizes of trunk mains needed to serve only the urban consumers. The total dollar value of the part of the trunk main system devoted to the service of suburban consumers was derived from a comparison of the cost of this system with that of the existing system, serving both urban and suburban consumers.

The fixed charges on this total dollar value of trunk mains assignable to suburban service represented most of the variable cost of transporting water to the metering stations serving the wholesale purchasers of water for surburban use. The distribution of these charges was accomplished by means of a hypothetical trunk main (Fig. 1) serving only the suburban points of purchase and laid out along the route of certain existing trunk mains around the outer edge of the Kansas City system. The total estimated cost of the hypothetical main was apportioned on the basis of a summary of determinations made of each suburban demand's share in each section of the main, and the apportionment thus arrived at was applied to the previously calculated total dollar value of trunk mains assignable to suburban service. The cost of a demand of 100,000 cu.ft. per day was then fixed for each point of suburban connection along the hypothetical main.

The charge for transportation from the hypothetical main—which coincided in fact with an existing trunk main—to the point of purchase remained to be fixed. The metering stations are connected to the hypothetical-main location through 6–16-in. lines and are roughly ½–2 miles from it. This added reach of service was figured into the distance-demand rates at the uniform cost of $450 per year per 100,000 cu.ft. of daily demand. This amount may appear somewhat low at present prices but it conforms fairly well to the cost of the older mains presently serving Kansas City's customers.

Most of the purchasers for suburban use have found it possible as well as advantageous to avoid the excess-hourly-demand charge by relying on storage to meet peak-hour requirements. Without this charge, the average cost of water for a year at any single point of purchase is found to be a function of the ratio of the maximum to the average daily demand and the daily-demand charge. As previously indicated, the range of these ratios for the Kansas City system is between about

140 and 250 per cent, while the daily-demand charges range from about $400 to $1,200 a month per 100,000 cu.ft. of daily demand. The yearly average cost per 100 cu.ft., based on the daily-demand charge and the maximum-to-average-day ratio, and excluding the excess-hourly-demand charge, may be found in Fig. 2.

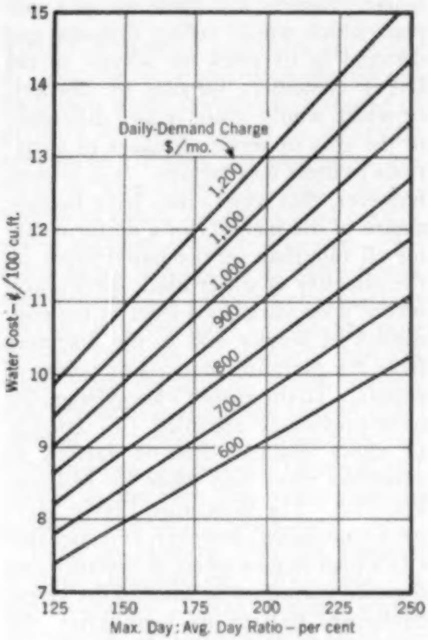

FIG. 2. Annual Average Water Cost

—was to the purchaser nearest the North Kansas City pumping and purification plant. The point of highest unit cost—averaging 10.25¢–12.80¢—was at the far southeast corner of the city.

The advantages of the rate were quick to assert themselves. Overhead storage, never popular in a residential

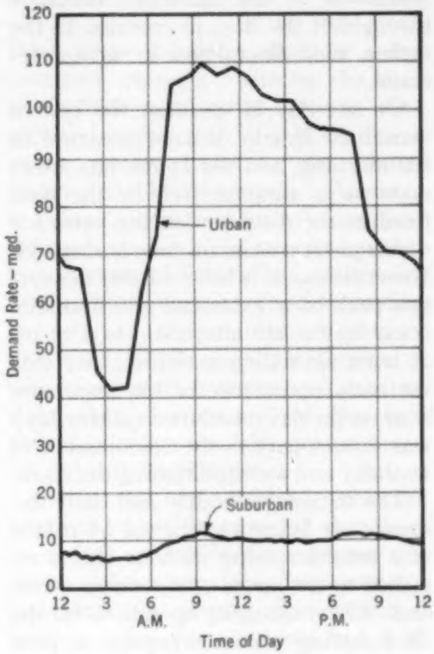

FIG. 3. Maximum-Day Urban and Suburban Demands

Effects of Plan

Kansas City has had two summers' experience with the distance-demand rates but apparently neither of them has included a dry period of the kind likely to cause the highest possible ratios of maximum to average days. Experience with the rate has indicated, however, that it results in an equitable distribution of the total cost of service to suburban customers. In the fiscal year ending April 30, 1950, the lowest unit cost of water—averaging 7¢ per 100 cu.ft.

area, was provided on the west side of the system for a section that would otherwise have preferred to rely on Kansas City's pumping stations and trunk mains for its peak-hour requirements. The high maximum-to-average-daily-demand ratio for golf courses was brought to light. Over a year this ratio, in the Kansas City area, is on the order of 350–400 per cent. One purchaser, passing on to two courses a pro rata share of the demand charges,

has caused them to exercise better control over their water use.

One quite beneficial effect of the rate on the hourly demands of suburban consumers is shown in Fig. 3, which compares the total suburban to the total urban demands for the day of maximum system and maximum suburban demand in 1949. It is interesting to note the steadiness of the suburban demands throughout the day, in contrast to the rather wide fluctuation in urban demand.

On the day in question the system maximum hourly demand occurred in the morning, and not in the late afternoon as is contemplated by the time fixed in the distance-demand rates for a charge on maximum hourly demand. Nevertheless, it is believed that the system peak hourly demand will normally occur in the late afternoon, as a result of lawn sprinkling activities, and that the early occurrence of the maximum hour on the day mentioned (a Monday) was due in part to the combination of washday and air-conditioning demands.

The maximum hourly and daily demands are being ascertained by means of a recorder using 24-hour charts attached to the hands of the water meter dial. Chart-changing operations for the 28 metering stations require a little more than the time of one man during the summer months.

Kansas City serves fewer than 100 suburban customers through meters $\frac{5}{8}$ to 2 in. in size. For such small customers whose maximum daily consumption is below a certain point, the use of distance-demand factors in determining the cost of service is not justified. Consequently, the suburban demand rate schedule includes a provision limiting its application to purchasers with a maximum daily demand of not less than 10,000 cu.ft. For the smaller customers, a surburban meter rate is employed, based on commodity cost alone.

Conclusion

Kansas City's distance-demand rates were formulated without the help of any precedent in the water works field. The author found nothing in the literature to indicate that any other city-owned water works property had made use of a rate plan which would reflect distance and demand in its rates for service to the larger customers outside its borders, or which would result in any difference in the cost of service because of variations in these cost factors. It is known, however, that some cities have become aware of the inequity of a uniform rate for all suburban service based solely on the quantity of water used, particularly where wide variations exist in the conditions of service and in the distances from the pumping stations or points of supply. To the author's knowledge, the only previously recorded (1) attempt to apply distance-demand factors to suburban rates was made by Milwaukee, Wis. The Wisconsin Public Service Commission, however, rejected that city's plan to give effect to distance and demand variations in rates to the larger customers outside its boundaries, the proceedings in the case lasting more than six years.

The author is convinced of the equity of a variation in rates for service to suburban customers in proportion to the variations in the cost of that service and believes, on the basis of two summers' experience, that Kansas City's distance-demand rate plan is workable.

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Charges for Private Fire Service

By Richard H. Ellis

A paper presented on May 25, 1950, at the Annual Conference, Philadelphia, by Richard H. Ellis, Hydraulic Engr., Eng. Div., Associated Factory Mutual Fire Insurance Cos., Boston, Mass.

FIRE protection engineers are today placing more and more dependence on public water systems as a source of primary or initial supply to automatic sprinklers. Where the properties are large or where there is a high concentration of value, independent secondary water supplies are also provided. Because of the divergence in water works requirements for private fire protection service, it is essential for the fire protection engineer to become familiar with local regulations when making new layouts for the protection of commercial or industrial properties, or when making suggestions for ways and means to improve the water supplies available at existing properties.

Early in January 1949 the Factory Mutual Engineering Div. sent, to approximately 1,500 water works throughout the United States and Canada, requests for information about their requirements and rates for private fire protection. In addition, two general questions were included which were designed to gather information indicating how many water purveyors (public and private) are reimbursed by their respective municipalities for public fire protection service and the percentage such revenue bears to total receipts.

The water works officials to whom these questionnaires were addressed cooperated in an excellent manner. With some follow-up, and, in a few instances, through visits by the division's field organization, returns were obtained from a total of 1,269 different water works, 1,158 in the United States and 111 in Canada. Answers, in general, were surprisingly complete, although, as might be expected, a certain amount of license had to be used in the interpretation of a few replies. In view of the excellent assistance given the author in the collection ot this information, it seems no more than right that a summary of the data should be made available to the members of this Association.

From the information presented herewith, it will be apparent that there is a lack of uniformity among the water utilities regarding requirements and rates for private fire protection service. To one who has had both water works and fire protection experience, this situation does not appear to be altogether the fault of the water works engineers and operators, as it is evident that there are many misconceptions about the relative water requirements for the protection of sprinklered and unsprinklered properties.

It is recognized that the responsibility of the water works operator is principally the management and operation of a utility, the purpose of which is to provide water for domestic, commercial and industrial use and for gen-

eral fire protection purposes. Usually water works people are well acquainted with the public fire protection requirements established by the National Board of Fire Underwriters. They are also cognizant of the claims of the fire protection interests that less water is required for the protection of sprinklered properties than of unsprinklered properties. Isolated examples have been quoted to substantiate these claims, but too frequently such information has been offered only in an endeavor to bring about a change in local regulations or rate structures and has therefore savored of prejudice. Seldom has any attempt been made to present factual data which would furnish the water works interests basic information on the relative demand for water for the extinguishment of fires in protected and unprotected properties.

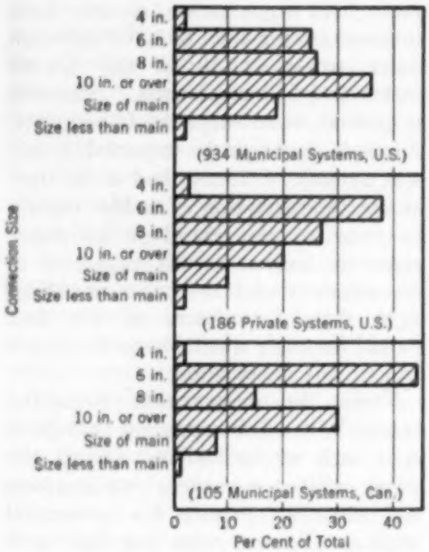

Fig. 1. Size of Connection Allowed

To clarify some of the apparent misconceptions, the author, in discussing the results of the Factory Mutual survey, will comment on some of the differences between the water works and fire protection viewpoints and will present the results of an original study conducted to determine the relative water demand created by fires in sprinklered and unsprinklered properties.

In presenting a summary of the data furnished by the water works officials, reference will be made to the prevailing practices of publicly and privately owned water utilities relative to [1] the maximum size of connections allowed, [2] meter requirements, [3] annual charges and [4] the basis of the charges.

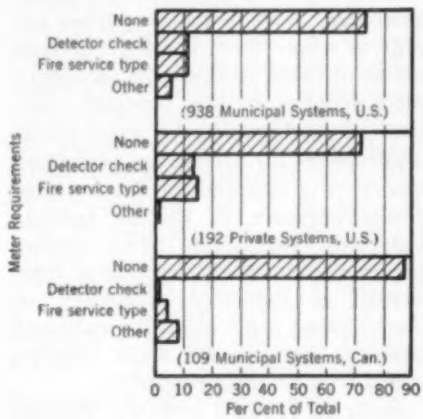

Fig. 2. Meter Requirements

Size of Connections

Figure 1 graphically illustrates current water works practice as it refers to the maximum size of the connections allowed for private fire service. It will be noted that about 2 per cent of the private and municipal water works, both in the United States and in Canada, limit the size of the connections to 4 in. In general, municipal systems

permit the use of 6-, 8- and 10-in. connections, and the practice in private systems is much the same, except that there is a somewhat greater tendency to limit the size to 6 in. The trend during the last ten-year period has apparently been to liberalize to some extent the limitations on the size of the connections, a greater percentage now permitting 8-, 10-, 12-in. or over as compared with the practice in 1939.

The fire protection engineer's interest in the size of the connection to private protection systems is primarily to minimize, as much as possible, the friction loss in the connection. To assure effective delivery from a sprinkler system, it is usually necessary to maintain 15–25-psi. flowing pressure at the top of the sprinkler risers. Frequently pressure is at a premium and any undue loss may reduce the effectiveness of the sprinkler protection. On the other hand, water works interests have sometimes held that it is desirable to limit the size of the connections to private protection systems in order to prevent undue reduction of pressure in the public mains in the event of a fire. The inconsistency in this contention appears to be that, if an ample supply of water is not initially available to the sprinklers, an amount at least equal to the sprinkler demand, or more, will be required for hose streams.

Meters

The installation of commercial types of meters on private fire service connections also introduces friction, and the expense of fire service meters is rarely justified. Practically all new sprinkler installations require the provision of flow alarm devices, which are often connected to a central station supervisory service. There are numerous other means by which a water utility can supervise fire protection connections. Except in large industrial properties involving a number of buildings with a considerable amount of yard piping for separate or combined domestic and process purposes, it seldom appears necessary to the fire protection engineer to provide meters. Where local conditions justify meters, however, the fire protection interests are particularly anxious that the metering device be of a type which will furnish an unrestricted waterway and will introduce a minimum of friction. These requirements are best met by detector check valves (metered bypass) or full-flow fire service type meters.

Figure 2 illustrates the present meter requirements for private fire service. It is interesting to note that a majority of the water works recognize the facts of the situation and do not require meters. Where meters are required, detector checks or fire service type meters are generally specified (except in Canada). Water works practice in this respect is somewhat more liberal than it was ten years ago. A greater percentage of those water works that do require meters are now specifying a type which is satisfactory from a fire protection viewpoint.

Private Fire Protection Rates

Probably the most controversial subject in discussions on private fire protection is whether or not the water purveyors should make periodic charges if private fire protection systems are connected to and supplied, in whole or in part, by public water systems.

Returns from the recent survey indicate that 56 per cent of the municipal systems levy no charge for private fire service connections (Fig. 3). Practice in Canada is a little more liberal, in

that 65 per cent of the water works there make no charge for sprinkler connections. On the other hand, privately owned works generally do charge for private fire service connections, but, as will be shown later on, there may be some justification for this since a large number of the private systems assume part or all of the cost of the connection to the property line. There has been relatively little change in water works practice in this respect, the percentage of municipal plants now making no charge being 4 per cent greater, and the percentage of private plants 1 per cent less, than in 1939.

Figure 4 shows the range of annual charges by municipal and private water

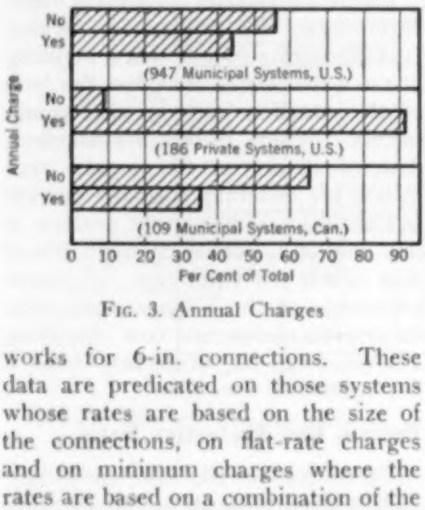

FIG. 3. Annual Charges

works for 6-in. connections. These data are predicated on those systems whose rates are based on the size of the connections, on flat-rate charges and on minimum charges where the rates are based on a combination of the number of sprinklers and hydrants and the size of the connection. Of those municipal works which do levy charges on such bases, 57 per cent charge $50 or less per year for a 6-in. connection. A very small percentage have charges that range from $200 up to a little less than $500 a year for each 6-in. connection.

Private practice varies from municipal practice in that only 16 per cent of the charges for 6-in. connections are $50 a year or less per connection and 61 per cent are between $50 and $201 per year. One water works company charges more than $500, receiving as much as $960 per year for each 6-in. connection.

Although the usual charge per sprinkler head is between 1¢ and 10¢

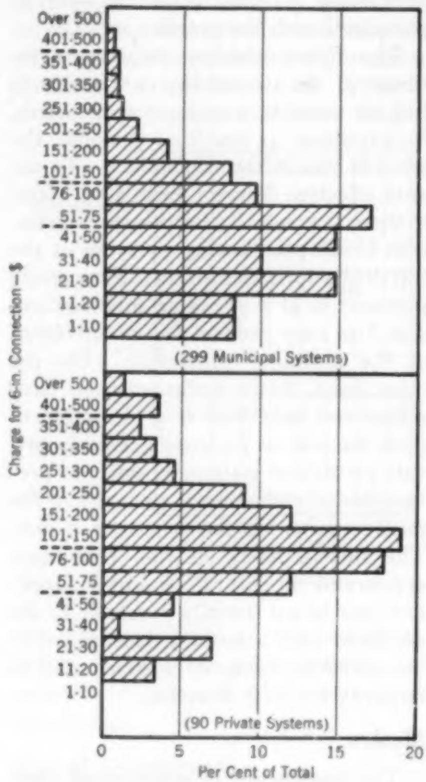

FIG. 4. Charges for 6-in. Connections

a year, some of the new rate schedules establish charges as high as 18¢–20¢ per head. Relatively few works charge more than 20¢, but there are reports of charges as high as 36¢ and even 63¢ per head.

Figure 4 indicates that there is very little uniformity in the charges for

private fire service levied by municipal works or private companies. How can one justify the fact that 56 per cent of the municipal systems make no charge and others charge nearly $500 a year per 6-in. connection, or that, in privately owned systems, nearly 10 per cent make no charge and one charges as high as $960 per year for the same service? Certainly the amount of the charge is no index of the ability to render service, as is evidenced by the

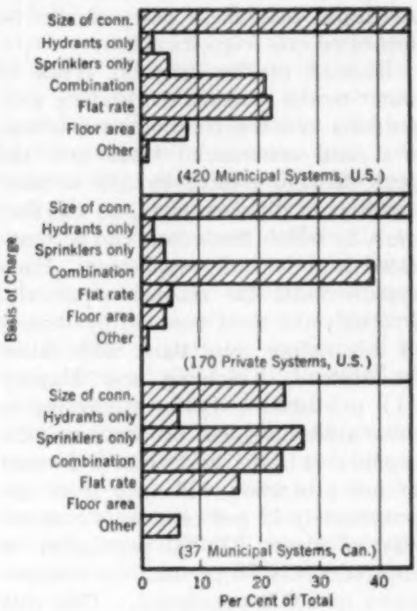

FIG. 5. Basis for Charges

fact that in some of the locations where the rates are highest the water supply is entirely inadequate from a fire protection viewpoint.

The same lack of uniformity again appears when the bases for the charges are studied.

Basis of Charges

In both public and private practice, 45 per cent of the water works in the United States and 22 per cent of those in Canada base their charges for private fire service on the size of the connections (Fig. 5). A few works charge on the basis of the number of hydrants only, and approximately 5 per cent of all works in the United States charge on the basis of the number of sprinklers installed. In Canada, the more general practice seems to be to base the charge on the number of sprinkler heads, this basis being adopted by approximately 26 per cent of all works charging for private fire protection connections. An appreciable number of works make charges

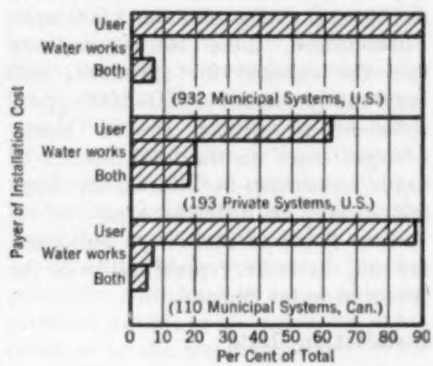

FIG. 6. Payer of Installation Cost

based on some combination of the number of sprinklers and the number of hydrants, standpipes and so forth. In municipal practice, both in the United States and in Canada, numerous rates are predicated not on the size of the connection nor on the number of sprinklers involved, but on a flat charge for any size of connection. Floor area is the basis on which a number of the municipal and private plants (relatively few) collect for this type of service.

The bases of the charges adopted by many of the water works authorities appear illogical to the fire protection

engineer. If charges based on the number of sprinklers installed, or on any combination of sprinklers and hydrants or on floor area, are intended to reimburse the water purveyor for the potential demand that might result from the presence of the protective equipment, it is evident that the basis is in error. Studies by the author indicate that the proportion of the sprinkler heads which would open in any one area as a result of a fire in a sprinklered property is not dependent upon the number of heads therein but upon the occupancy, construction and square-foot area involved, and that it is highly improbable that all heads will open in any area in excess of about 5,000 sq.ft. Furthermore, these studies indicate that the demand for sprinklers will very rarely exceed 1,500–2,000 gpm., sufficient to supply 75–100 heads. Charges based on the total number of heads, sometimes in different buildings and usually in different areas, or on various floors of multistory buildings, are not, therefore, representative of the potential water demand.

Installation Costs

When an application is made for a new fire service connection, the applicant is usually asked to make a deposit to cover the cost of installation. In 90 per cent of the municipal systems, the installation costs are paid by the applicant; in 3 per cent, by the water works; and in 7 per cent, by both the water works and the applicant (Fig. 6). The private companies, at least on the surface, appear to be more generous, in that 20 per cent bear the entire cost of the connection and 18 per cent bear part of it. As previously mentioned, this, of course, does justify a charge based on the amount invested and on resulting maintenance and depreciation costs. The Canadian practice follows very closely that of the municipal systems in the United States.

Public Fire Service Charges

The two principal functions of a water works are: [1] to provide water for domestic, commercial and industrial use; and [2] to provide water for fire protection. The expenses involved in the production of these supplies must, of course, be recovered, and it is logical, in the development of a rate structure, that these costs should be separated one from the other.

Because of the different types of water works systems, the varying proportions between the domestic, industrial and commercial loads and the peak demand created by fire in communities of different sizes, no one formula by which these costs can be separated is universally applicable. Each system must be studied separately. Probably the most noteworthy studies of this nature were those undertaken by Metcalf, Kuichling and Hawley (1), published in 1911. According to these authorities, the proportion of the capital cost of the works that is devoted to fire protection will vary from approximately 13 per cent for a community of about 300,000 population to approximately 70 per cent for communities of 5,000 population. This cost must be collected if the works are to be self-supporting, and it is, therefore, a fallacy to consider that any public or private water works whose receipts balance expenses is furnishing "free" water for fire protection. The question of how the increment of cost allocable to fire protection service should be collected is one that is troublesome to the water works authorities. In theory it would appear equitable to collect for public fire protection service an

amount in proportion to the value of the individual properties protected. The most direct way of doing this would, of course, be for the municipality to appropriate and pay to the water works an amount equal to the cost of providing fire protection and to collect it from the public as a part of the tax levy.

Such a procedure is actually followed to some extent in 36 per cent of the municipal systems in the United States, 64 per cent of the systems in Canada and 89 per cent of the municipalities which are served by private companies (Fig. 7). This recognition of the principle that fire protection is a governmental function and should be paid for by the municipality is encouraging. It is, however, a fact that only a few municipally or privately owned water works are reimbursed sufficiently to cover the probable cost of furnishing this service. The natural hesitancy to add additional increments to present tax burdens is probably detrimental to the general adoption of this method of collecting fire protection costs.

In at least one known instance the water bill is divided into two parts—one to cover the amount of use and the second, based on a rate expressed in mills per dollar of assessed property value, to cover public fire protection cost.

It is generally recognized that a water works should be self-supporting. Rates, whether for water sold, service rendered or both, should be predicated on the expectation that the total collections will cover all operating and capital costs. In those works where receipts balance expenses but where no contributions or only token payments are received from the municipality for fire service, it is evident that costs allocable not only to the production and sale of water but also to the provision of fire service have been included in the rate schedule. Although such a procedure may furnish ample revenue, an apparent inequity results in that payment of the fire protection costs is prorated on the amount of water consumed rather than on the value of the property protected.

Many of the municipally owned plants pay no taxes and receive no direct revenue from the municipality for public fire protection, and it is occasionally contended that the waiver of taxes offsets fire protection costs. Seldom, however, does this constitute an exchange of equal value. Either the community or the water department loses. For instance, if the tax value exceeds the cost of providing water for fire protection, the property owner must stand the loss. On the other hand, where the tax value is less than the water department's expenditure for fire protection, the latter must look to the water consumer for the value so lost. In either event, a portion of the fire protection costs must be assumed by the property owner, as an increment in his taxes, or by the water consumer, in his rates. There is no such thing as "free" water for fire protection.

Attention has already been called to the fact that 36 per cent of the municipal plants are directly reimbursed to some extent for public fire service by the municipality. About 24 per cent of all municipally owned works receive revenue from both private and public fire service. Of the private water works, 89 per cent receive revenue for public fire service and 80 per cent receive revenue from both private and public fire service. A little less than half of the Canadian water works which are reimbursed for public fire service also receive revenue from private fire service.

Unless it can be established that the water works is rendering the owner of privately protected property a special service in addition to that furnished to the community as a whole, it would seem that the owner of sprinklered properties who is bearing his share of the cost of public fire protection (or a portion thereof) in his taxes or rates is to some extent being charged twice for the same service. The justification for charges for private fire protection appears to hinge on whether or not a special service is rendered.

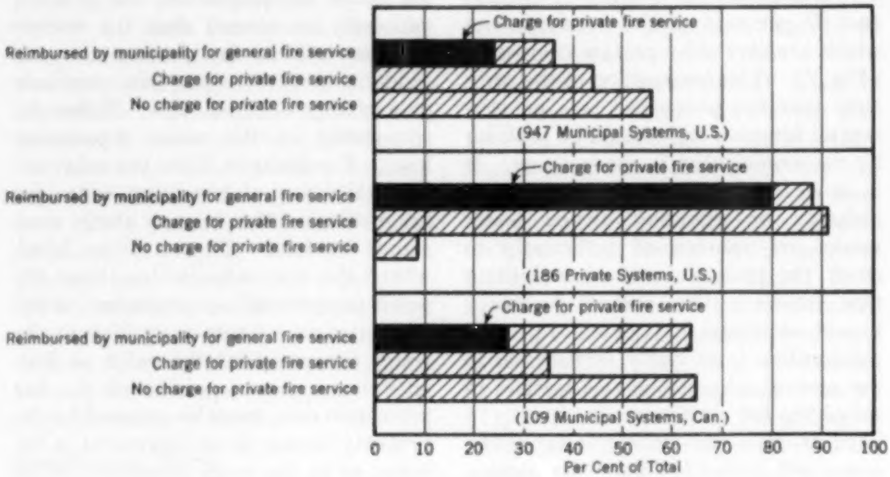

Fig. 7. Public and Private Fire Service Charges

Sprinklered and Unsprinklered Demand

The fallacy of the contention that a special service is always rendered by the water works to owners of sprinklered properties appears to be proved by a comparison of the water demand for fires in sprinklered and unsprinklered properties of the same general type of occupancy (Fig. 8). The curves shown have been developed from National Fire Protection Assn. and Factory Mutual records and cover fires which occurred in cotton storage, cotton textile processing, flammable-liquid, machine shop and baled-wastepaper occupancies. As occupancies of the types studied are generally sprinklered, a search of the N.F.P.A. records covering the period 1937–49 furnished only 92 instances where there was definite evidence of the water demand in fires occurring in unsprinklered properties so occupied. The water demand at these fires is represented by Curve *A* in Fig 8. Curve *B* has been plotted from the 1940 Factory Mutual record of 621 fires in sprinklered properties with similar occupancies. Although the data on unsprinklered properties are limited, comparison of these two curves furnishes glaring evidence that the water demand in sprinklered properties is but a small portion of that which would result from fires in unsprinklered properties with the same occupancies.

It is quite evident from studies of this nature that, if a water system is properly designed to furnish public fire pro-

tection service such as would be required for the extinguishment of fires in unsprinklered properties, the same service will be much more than that needed for the supply required in protected properties. For instance, only 15 per cent of the fires in sprinklered properties required 500 gpm. or more for both automatic sprinklers and hose streams, as compared with practically 100 per cent of the fires in unprotected properties. Similarly, only 1 per cent of the fires in sprinklered properties required 2,000 gpm. or more, whereas 60 per cent of the fires in unsprinklered properties required the same amount or more. The maximum demand in unsprinklered properties was about 7,500 gpm., as against 4,500 gpm. in sprinkler-protected properties.

Although there are various arguments offered to justify charges for private fire protection, the author is of the impression that, largely because of the lack of reliable data, some water works men feel that an automatic sprinkler system creates an additional load on the water works plant and that the demand by sprinklers requires a special service in addition to public fire protection service. This contention is not borne out by the author's studies.

How can any portion of the total water works cost of providing general fire protection be allocated to private fire protection service if the latter creates less of a demand than the former and involves no additional cost to the water works? From the viewpoint of the fire protection engineer, such a cost division is impossible of accomplishment and many of the water works authorities apparently agree, as evidenced by the fact that 56 per cent of all municipally owned water utilities make no attempt to collect anything for private fire protection. The other 44 per cent may levy their charges on the assumption that they are providing a special service, but there seems to be no definite or fixed method by which they can support this theory or separate the cost of public and private fire protection service.

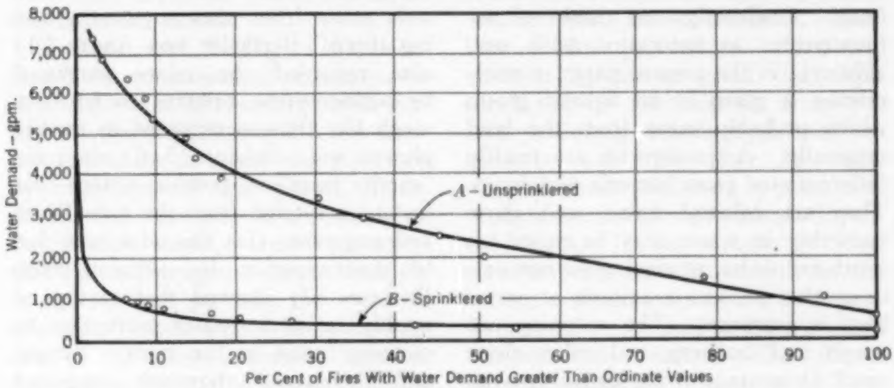

Fig. 8. Sprinklered and Unsprinklered Fire Demand

Reference

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Actinomycetes and Common Tastes and Odors

By J. K. G. Silvey, James C. Russell, David R. Redden and William C. McCormick

A contribution to the Journal by J. K. G. Silvey, Chairman, Science Div.; James C. Russell; David R. Redden; and William C. McCormick; all of North Texas State College, Denton, Tex.

THE actinomycetes, or ray fungi, are a group of organisms that are closely related to the bacteria and the fungi. Ordinarily, one thinks of actinomycetes as associated with soil, although, in the present paper, consideration is given to an aquatic group which probably came from the land originally. Actinomycetes are readily differentiated from bacteria and fungi. They are colonial forms, and those occurring in water may be raised on synthetic media, producing surface colonies that present a smooth or wrinkled appearance. The colonies are tough and leathery and often show small filaments growing high above the media. The actinomycetes may look like true bacteria in certain laboratory preparations because they fragment easily and some of the reproductive stages resemble bacteria cells. In the observation of unstained organisms, the branching and spore formation may resemble a true fungus. Actinomycetes may produce "earthy" odors in laboratory culture, and, in liquid media, the products of their normal growth can be isolated for chemical studies.

Over 50 years ago Rullman (1, 2) described a type of actinomycete that produced a pungent odor which he described as "earthy." Biejerinck (3) had the same opinion and concluded from his work that possibly some of the odors freed from newly tilled soils arose from actinomycetes growing there. Berthelot and Andre (4) also regarded the odors produced by actinomycete cultures to be very much like that encountered in freshly plowed soil. Adams (5, 6) cited the "earthy taste" in potable waters that had been filtered from the Nile River and suggested that the odor was due to contamination by actinomycetes. Rushton (7) showed that earthy or muddy tastes in water were due to diatoms found in the river. Burger and Thomas (8), however, suggested that, in a water studied by them, the earthy taste was doubtless the result of complex microbiological decomposition of plants growing in the rivers from which the samples under investigation were being drawn. They did not deny that actinomycetes could impart "earthy tastes" to water but preferred to conclude that the taste resulted from the activity of a mixture of microorganisms.

Thaysen (9) became interested in this subject because salmon caught in some of the richest salmon rivers in England were rendered unpalatable by the earthy or muddy taste of their

flesh. In his investigations, he found that actinomycetes were the cause of the earthy or muddy taints both in the water and in the fish using the water in respiratory processes. He also isolated an organic compound which was slightly soluble in water, volatile in steam, soluble in ether and partly soluble in alcohol. In concentrated form, the substance assumed a brown, amorphous appearance and produced a penetrating manurial odor. The same substance, dissolved in water in 0.2-ppm. concentrations, imparted a pungent, earthy taint like the one Thaysen had observed both in the water and in the fish from the stream.

Umbreit and McCoy (10) mentioned the occurrence of actinomycetes of the genus *Micromonospora* in certain northern lakes, but did not comment about the taste and odor produced by organisms of this genus. Issatchenko and Egorova (11), working on the Moscow River, were able to show that the actinomycetes which multiply in the silt and get into the water in relatively large quantities constitute one of the most important agencies responsible for earthy and unpleasant flavors in the water. They assumed that the actinomycetes were land forms which had adapted themselves to an aquatic mode of existence. They further attributed the luxuriant growth of these organisms to vegetable matter and sewage contamination, a combination of which served as nutrients for the actinomycetes.

Earthy, muddy, musty, woody and potato bin odors are common in the rivers, streams and reservoirs of the Southwest. The senior author has observed outbreaks of one or more of these odors and tastes in most of the rivers and reservoirs of the region, beginning in April and continuing to November. The first concentrated effort toward investigating these phenomena took place during 1938–42 at Lake Dallas, a reservoir which serves Dallas, Tex., as a water supply. It was then the impression that a diatom, *Melosira*, was responsible for the abnormally high threshold odors found in the water, even though the concentration of odor was much greater in the bottom deposits than in the water above.

In 1946 an investigation was made on a new series of reservoirs, some of which were high in odors in warm weather while others exhibited no offensive tastes and odors. After two years it became apparent that algae could not be the culprits, since the lakes with large numbers of these organisms had little taste or odor, whereas those producing a poor crop of algae frequently showed high threshold values. In the summer of 1948 actinomycetes were isolated from the bottom mud of some of the odor-producing reservoirs and a pure culture was ultimately obtained. The cultures were presented to Waksman at Rutgers University for identification, and he stated that they belonged to the genus *Streptomyces*. The species has not yet been determined.

Surveys of a number of streams and reservoirs in the Southwest have uncovered *Streptomyces* in their bottom deposits. Many cultures have been raised in the laboratory and their by-products have been examined chemically. At present it is possible to state that the major by-products produced in normal culture of these organisms are saturated fatty acids, an unsaturated aromatic compound and amines. The amines appear to be of at least two

types, isoamyl amine and isobutyl amine. It is probable that other amines are also produced, the types depending on the kind of nutrients furnished the actinomycetes, as well as on the temperature and dissolved-oxygen content. The saturated fatty acids are several low-molecular-weight compounds, the major ones being valeric acid, isovaleric acid, β-hydroxybutyric acid and isovaleraldehyde. The unsaturated aromatic compound appears to have either an aldehyde or an acidic group attached. This compound in normal hexane shows a maximum absorption at 375 mμ and has a molecular weight of 510 ± 2 per cent.

Environmental Conditions

The actinomycetes of the genus *Streptomyces* found in Southwest reservoirs are especially responsive to changes in temperature. The minimum temperature at which the spores will germinate and produce a vegetative growth is 15°C. At this temperature, few of the by-products of the organism are apparent. At 17°C., the by-products may be extracted either from water or from a culture medium, although the total concentration of chemicals will be minimal. As the temperature increases, the activity of the organisms is enhanced, and, so far as is known, the maximum temperature for growth stimulation is not exceeded in the natural waters of this section of the country, even though readings of 38°C. (100°F.) have been recorded from shallow areas of reservoirs during August.

Light is apparently an unimportant factor, since the organism has been found in both clear and muddy lakes. In culture media, no difference in by-products has been observed when actinomycetes are reared in the light or in the dark. Surveys on reservoirs show that actinomycetes grow best in water 6–11 in. deep and least readily in lakes 40 ft. or more in depth. It is, therefore, apparent that shallow lakes or those with considerable shallow areas will produce the organism in greater concentration than will lakes with plummet basins.

The concentration of *Streptomyces* in reservoirs is in direct proportion to certain types of available organic matter. Polysaccharides, proteins, peptids, amino acids and undoubtedly other complex organic compounds serve as nutrients for these organisms. In reservoirs with considerable areas of shallow water supporting a rich growth of cattails, saw grass, eelgrass, bulrushes, water lilies, button willow and black willow, one may find luxuriant growths of actinomycetes during the warm season. Where there is a dearth of the abovementioned types of vegetation, it is likely that algae serve as nutrients for the actinomycetes. Thus, the higher the concentration of planktonic and attached algae, the greater the growth of *Streptomyces*.

If the water is alkaline, the actinomycetes grow in quantities, and the greater the alkalinity, the more prolific the growth. In like fashion, the mineral content supports the growth of these organisms. In the eastern section of Texas where impoundments occur on acid sands, the growth of actinomycetes is generally poor and the amount of odor in the water may be negligible. These conditions may, however, be modified by copious growths of littoral vegetation or algae.

Taste and Odor Production

The odors and tastes produced by *Streptomyces* may be attributed to the simple and complex amines, saturated

fatty acids and unsaturated aromatic compound which are by-products of the metabolism of these organisms. As previously mentioned, the concentration necessary to produce a noticeable taste and odor in water is approximately 0.2 ppm. If the concentration is increased, the odors and tastes become more apparent and may appear manurial during the maximum heat of the summer.

The different odors and tastes produced by these organisms vary, depending on the treatment given to the water. If prechlorination is practiced to provide both a combined and free residual chlorine, as has been demonstrated on several water supplies containing actinomycetes, a breakpoint may be encountered after 10 ppm. has been added. Samples used in the breakpoint series contain both combined and free residual chlorine although the proportion of combined to free is least at 10 ppm. The total residual chlorine in the sample to which 10 ppm. chlorine had been added was 3.2 ppm., with the free residual chlorine comprising 3 ppm. Since the threshold odors rise with each addition of chlorine, it is apparent that actinomycete by-products act differently from other chloroderivatives thus far studied in water supplies.* The characteristic odors exhibited by these chloroderivatives may be described as chlorinated "muddy-fishy," "earthy-marshy," "earthy-manurial," "w o o d y-musty," "rotten stump" or "woody-fishy." These descriptions are inadequate, but they may serve to illustrate the different odors that can be noticed when chlorine comes in contact with the various compounds.

Additions of activated carbon have proved more successful than chlorine in many areas. For example, in 1937–38 Dallas used as much as 55 ppm. activated carbon in an attempt to combat the odor produced by actinomycetes in the city water supply. Even this amount did not remove all of the odor, but it reduced it considerably. In all probability, if the purification plant effluent was relatively free of odor and taste, it picked up additional tastes in the distribution system, as will be explained in the next section.

Chlorine dioxide has been found relatively effective in eradicating the taste and odor, provided chlorine is not used in sufficient amounts to produce a total (free and combined) chlorine residual of as much as 1 ppm. Since the chlorinated by-products require five times as much chlorine dioxide for oxidation as the by-products alone require, the chlorine dioxide demand is obviously increased if chlorine is employed.

It is likely that other compounds would be efficient in removing the compounds produced by actinomycetes, but thus far none of those tested appears to be effective. Ozone, bromine and oxygen have been tried on a laboratory scale, with little or no success. It is possible that fluorine compounds would be useful if a method could be devised to produce the oxides of fluorine in an economical and safe fashion. Certainly, this problem deserves considerable investigation, since the authors' research shows that most of the tastes and odors in rivers, streams,

* The increase in odor and taste intensity was well brought out by Howard and Thompson (12), but at that time threshold odor methods were not yet established. It is now known that this condition always obtains *prior to* the production of residuals which are predominantly free available chlorine, and that the minimum threshold odor values will occur only *after* such residuals have been maintained.—*Referee's note.*

natural lakes and reservoirs throughout the Southwest are produced by actinomycetes and not by algae of themselves.

Detailed Studies

Detailed studies have been made with chlorine in an attempt to remove the actinomycete by-products from water by prechlorination. Only recently the water supply in Ranger, Tex., became contaminated with a growth of actinomycetes that produced compounds in concentrations of 0.5–1.6 ppm. Prechlorination tests on this water, using 1–20 ppm. chlorine, demonstrated that the original threshold odor of 251 on the raw water was increased to 601 in the sample containing the maximum concentration of chlorine.* The free residual chlorine in the last sample amounted to 6 ppm., and the combined residual chlorine was 14 ppm. In the sample receiving 10 ppm. chlorine, a free chlorine residual was established which comprised 90 per cent of the total residual chlorine. Even though a breakpoint was graphically present, the odor and taste of the treated sample were greater and more offensive than those of the raw water. On the other hand, chlorine dioxide used in like samples of raw water reduced the threshold odor from 251 to 6.0. The amount of chlorine dioxide added to the sample exhibiting a threshold odor of 6.0 was 9 ppm. and the residual was 3.2 ppm.

Observations in both field and laboratory for the past ten years have demonstrated that the actinomycete-produced odors described in this paper are not reduced by chlorine in either a combined residual or a free residual form. As stated above, even when the free chlorine residual was 90 per cent of the total chlorine residual, it failed to oxidize the combined residual chlorine. Thus, it is apparent that the chloroderivatives formed are not subject to the complete oxidative action of the free residual chlorine in the water. It is possible that an application of over 20 ppm. chlorine might be more effective, although, above 10 ppm., in the samples tested, the total residual chlorine was equal to the chlorine added. If this is true, the point at which oxidation reactions are completed should have been reached.

Actinomycete Control

So far as is known, there is no chemical control for actinomycetes in rivers, streams or reservoirs. Copper compounds, chlorine and even arsenicals have been tried, with little or no success. Sodium chlorite in an applied concentration of 10 ppm. will eradicate actinomycetes of the genus *Streptomyces*, but this is an expensive procedure and probably not a wise practice, since all the water in the reservoir will not be consumed in one season. The authors, therefore, do not suggest chemical control of the actinomycetes in reservoirs but urge rather that all efforts toward taste and odor removal be made in the purification plant.

Natural control of actinomycetes not only is possible but can be readily accomplished if the engineering personnel cooperate adequately with the biologists in reservoir planning. If a reservoir can be constructed to contain a minimum amount of shallow water, and if all marginal and littoral vegetation is continuously sprayed or mowed,

* It is questionable whether the 20-ppm. maximum dose was sufficient. Severe conditions at Ottumwa, Iowa, required chlorine dosages up to 120 ppm. to achieve taste and odor control.—*Referee's note.*

the main nutritional sources for the actinomycetes are destroyed. This procedure will reduce or eradicate the amount of actinomycete growth throughout the shallow-water region. If algae blooms of unnatural proportions are reduced to a minimum, another source of nutrition for the actinomycetes will be removed. In other words, sewage and industrial pollution and other types of organic infiltration should be diminished or reduced to a minimum. An adequate and well supervised fisheries management program instituted on any reservoir should take care of natural algae blooms so that they will not be out of proportion to the normal requirements of the microorganisms comprising the aquatic microcosm. Complete control would result in little algae remaining in the water to serve as a nutritional medium for actinomycete growth. Successful programs can be prosecuted if the engineers, public health workers and fisheries biologists cooperate before the reservoir is constructed and after it has been established, to the end that an environment shall be established which is unfavorable to actinomycetes.

The spores produced by *Streptomyces* are so small that a large percentage of them may pass through a common bacterial filter such as a Seitz or Berkefeld. Thus, the average sand filter in a water purification plant does not constitute a barrier to the spores of these organisms. After the spores have passed through the filters and arrived in the mains of the distribution system, they may become attached to the inner irregular coating of mineral deposits and may produce normal colonies. The colonies elaborate their by-products, which results in new foci of odor and taste production.

The water in the main must attain a temperature of 17°C. (63°F.) before tastes and odors become noticeable. As the temperature rises, the amount of by-product increases until the water becomes unpalatable. If chlorine is added at the purification plant to produce either a combined or free residual, the threshold odor, as already mentioned, will rise in intensity, although there may be an alteration in characteristics of the odor and taste. For example, in a breakpoint series from Ranger, Tex., on March 27, 1950, it was observed that 5 ppm. chlorine produced the first noticeable chloroderivative odor. As the concentration of total available chlorine increased, the chloroderivative odor continued to intensify. At no time was the actinomycete taste and odor masked completely by that of the chloroderivative.*

Frequently a purification plant, through the use of carbon or chlorine dioxide, will produce an odorless, tasteless water which, because of the actinomycete by-products in the mains, will reach the consumer as a repulsive commodity, useless for cooking or drinking and frequently for laundry work. If cattle or chickens consume the water, the milk, eggs and flesh of the animals will carry the taint of the actinomycetes.

The problem of actinomycetes growing in the mains has been more or less

* As commented on the previous reference to the Ranger experiments, it does not appear that the total chlorine applied reached the magnitude that has elsewhere been found necessary to correct the most severe conditions. The experiments should have been continued with dosages at least up to the level employed at Ottumwa, where operating experience has shown the value of extreme chlorine doses to combat extreme conditions. —*Referee's note.*

overlooked in all previous investigations. The authors first observed it in Waco, Tex. Samples of tap water were collected at intervals between the purification plant and the termination of the mains in various parts of the city. The samples were run through a high-speed, continuous-flow Sharples Super Centrifuge,* which concentrated the spores for plating on nutrient agar. Counts demonstrated a continuous rise in actinomycete population from a point near the purification plant to the termination of the mains. Threshold odors rose in the same proportion as the colony counts from the samples.

Distribution systems deeply implanted in the earth may not be affected by the summer temperatures sufficiently to support a growth of the actinomycetes. Once actinomycetes are established on the mineral deposits in the mains during warm weather, however, they can be eradicated only with approximately 8 ppm. of chlorine dioxide. It is necessary to add this concentration to the total system once every five days. The reason for the periodicity is that the spores are not readily killed even though the vegetative colony is destroyed, and the spore incubation period is approximately seven days. Thus if, every five days, an 8-ppm. concentration of chlorine dioxide is applied to the distribution system, the by-products will not be produced, as the vegetative colonies will be absent. It is interesting to observe that the residual chlorine dioxide from such applications drops to 2.5 ppm. rather rapidly, and, in the terminal areas of the distribution system, the residual is 0.37 ppm.

* A product of the Sharples Corp., Philadelphia.

Evaluation

It is important, in an investigation of taste and odor, to isolate the organisms responsible for producing the substances liberated in the water. It is suggested that the investigator collect samples of muddy water from the basin of the lake, especially in regions where the depth of the reservoir will average around 10 in. during the warm season of the year. The samples should be collected in sterile, evacuated test tubes and plated out on Sabouraud's maltose agar. The characteristic growth of the actinomycete is recognized as whitish-gray scalelike colonies that appear to be slightly wrinkled and which grow to about 6 mm. in diameter. If several of these colonies appear on a plate, they should be transferred to other plates of the same medium until a relatively pure culture is obtained. Then transfers may be made to nutrient agar slants, where the culture can be kept active by making a transfer once every two weeks. The characteristic odor produced by the actinomycete can be detected readily by smelling the cotton plug. If the odor from the test tube is not similar to that found in the water of the reservoir, the investigator has failed to collect the actinomycete. (The reader is to understand that these directions are not intended for the expert but for all individuals interested in general water works operations.)

As indicated above, it is hardly practical to attempt chemical control in the reservoir. Distinct improvements can be accomplished, however, by destruction of all types of marginal and littoral vegetation. Algae investigations should be instituted on the reservoir and, if there is a bloom of algae above that normally expected, a sanitary survey should be made of the watershed

to determine if sewage or phosphorus is getting into the supply. The filtration plant and distribution system should receive immediate supervision in order to do as much as possible to remove the odor-producing compounds before the water enters the clear well.

Frequently the addition of soda ash in preparation of the alum floc is appropriate. A soda-alum floc will remove some of the by-product by a process of adsorption when a lime-alum floc is not effective. If prechlorination is practiced, it should be stopped for the time being and, if possible, chlorine dioxide treatment should be substituted. It is suggested that no more chlorine dioxide be added than will be consumed, especially if the operator desires to use activated carbon, because the latter has more affinity for chlorine dioxide or chlorine than for the chemicals which produce the odor. If chlorine were employed, any not consumed before activated carbon is added would also be wasted.

Activated carbon should not be introduced at the same time that the alum and soda ash are added but should enter at a point where the floc has settled out. The chlorine dioxide should be added to the clear well and occasionally, about every five days, a strong solution of it should be added to the backwash water going to the filters, in order to reduce the actinomycete products which apparently cling to the filter sand particles. If the operator feels inclined to add chlorine, it should be done only in the clear well and the concentration should not exceed the amount required to maintain satisfactory conditions within the water distribution system. Since chlorine dioxide is an efficient bactericidal substance, chlorine is not essential.

Careful scrutiny of the distribution system should be practiced at all times during the siege of actinomycetes. It is important to read the temperatures of the water in various parts of the distribution system. If the temperature rises above 17°C., or 63°F., samples should be collected for threshold odor tests. If a variation occurs in the threshold odor value within the distribution system, it is likely that there is actinomycete contamination in the mains. The only control known at present for this condition is the use of chlorine dioxide, already mentioned.

Conclusion

The data presented in this paper are intended to convey general information secured from investigating Southwest rivers, streams and reservoirs for a period of fifteen years. For several years the authors attempted to isolate substances from algae cultures which were reported to possess odors and tastes previously described in the literature. When the algae cultures were in good physiological condition, no chemical compounds were isolated which resembled the odor-producing substances often referred to by various investigators as "essential oils." It was finally concluded that the odors attributed to algae by many investigators were probably not produced by the algae, as previously assumed, but were produced from the algae by an intruding form. This intruder has been identified as a group of actinomycetes, *Streptomyces*, which is only one genus in a great array of organisms that may destroy not only algae but many types of aquatic and terrestrial vegetation and produce a myriad of chemical compounds.

The authors feel especially fortunate in having been able to isolate the genus discussed and believe that the recovery of compounds from both water and culture media will help to solve some perplexing odor and taste problems. Although the authors do not wish to go on record as stating that algae produce no taste or odor, sanitary engineers and others interested in water supplies of the Southwest must now concern themselves with organisms known to be responsible for tastes and odors, such as the actinomycetes.

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Possible Hazards From Chemical Contamination in Water Supplies

By C. W. Muehlberger

A paper presented on May 23, 1950, at the Annual Conference, Philadelphia, by C. W. Muehlberger, Toxicologist, Div. of Labs., State Dept. of Health, Lansing, Mich.

It is axiomatic that a safe and palatable supply of drinking water is essential for the maintenance of the public health. Not only is the human body composed chiefly of water, but water is the largest single component of the daily intake required for bodily nutrition and growth. Water supplies should therefore be of such a nature as to foster increased drinking; they should be free from unpleasant taste, odor or color, be bacteriologically clean, and be free from harmful chemical impurities. There has been much study and discussion concerning the control and elimination of unpleasant tastes or odors from drinking water supplies through aeration, chlorination or filtration. Objectionable color of the water supply itself, or its tendency to produce unsightly stains on sanitary fixtures, has likewise been controlled. The bacteriological purity of drinking water has been thoroughly investigated. Consequently, the author will address himself solely to the problem of the possible health hazards which may arise from chemical contaminants. Twelve years ago Sidney Negus of the Medical College of Virginia published a resumé of this subject in the JOURNAL (1). The present paper will serve to amplify his excellent presentation with more recent data and experience.

Absolutely pure water is almost unknown. Even the distilled water used in the chemical laboratory contains infinitesimal amounts of dissolved gases or solids. Furthermore, no one would care to drink pure water if it were available, because it is tasteless and "flat." So, from a practical standpoint, all water supplies carry appreciable amounts of substances other than H_2O. The problem is to consider the dangers to health which may arise from the drinking of water containing these foreign substances. To what extent may they have a poisonous action if drunk over a considerable period of time?

Toxic Limits

For most so-called poisons, there is ample information on the effect which may be expected if a person takes a single dose. The resulting acute poisoning may be very severe and may terminate fatally. But, when the subject survives, the poison will usually be eliminated eventually and he will recover. Toxic elements in drinking water, however, produce an entirely different situation. Instead of a single dose of the poisonous element, the body is exposed to smaller doses repeated many times daily over periods of months or years. If the toxic element is of such a nature that it is

eliminated from the body as rapidly as it enters, no accumulation will take place in body tissues. On the other hand, if the amount of toxic element taken into the body exceeds the body's ability to excrete it, there will be a gradual accumulation. Such storage may result in a mild and insidious type of chronic poisoning, hard to recognize or diagnose.

There is a wide difference in the rate at which the normal human body can eliminate various toxic elements. Naturally, a healthy individual might be expected to eliminate or detoxify a certain poison at a more rapid rate than a person who is ill and who perhaps has poorly functioning excretory organs such as the liver or kidneys. Likewise, adults can tolerate larger amounts of toxic elements than can young children or infants, a fact which is well illustrated by the effects of ingesting shallow well water containing appreciable amounts (usually over 100 ppm.) of nitrate nitrogen. Apparently, adults and even children can tolerate such concentrations of nitrate without difficulty, but infants fed on a formula prepared with this water frequently suffer from nitrite poisoning, resulting from the partial reduction of nitrate in the intestinal tract (2).

There is a wide variation in the tendency of various toxic elements to accumulate in the body. Some, such as zinc and copper, are eliminated easily and rapidly, while others, like lead or arsenic, have a far lower excretion rate, which means a greater tendency to accumulate in the body and thus to cause chronic poisoning. Finally, to complicate further an already complex situation, it is noteworthy that all persons are not alike in their sensitivity to poisons. Some individuals confronted with a certain toxic exposure in drinking water will develop symptoms of poisoning, while others in the same family will show no harmful effects whatever. With all these variables, it is difficult to establish limits for these chemical contaminants which will be safe and yet reasonable.

Obviously, it is of chief interest to answer the question: "How toxic are these agents to human beings *when taken daily in drinking water*?" Lacking data on this point, it is proper to inquire about their toxicity to man when taken in food, inhaled in the form of dusts or fumes, or injected experimentally. The latter observations, though valuable, may be misleading. Some toxic agents, when taken with food, are less readily absorbed from the gastrointestinal tract than when dissolved in drinking water. For this reason, they may be less dangerous as food contaminants than as impurities in drinking water. When poisons are injected experimentally or when they are inhaled in the form of dusts or fumes, absorption is necessarily complete, whereas, if taken in food or drink, an appreciable quantity of the toxic agent may be eliminated by way of the bowels and thus escape absorption.

Where information concerning chronic toxic effects on humans is not available, experiments on lower animals may provide guidance. Generally speaking, the higher animals, such as apes and monkeys, may be expected to offer more reliable inferences than the lower forms—dogs, cats, rabbits, guinea pigs, rats, mice, frogs and so forth.

Some of the most valuable information concerning toxic limits has re-

sulted from observations of persons who have used water supplies of a known degree of contamination over considerable periods. Thus, it has been learned from actual observation that drinking water containing 1 ppm. of lead eventually gives rise to symptoms of chronic lead poisoning. Consequently the accepted limit for the lead content of water has been placed at one-tenth of this amount (0.1 ppm.). By similar experience and observation, the limit for fluorine has been established at 1.5 ppm. Many observations of chronic arsenic toxicity resulting from contaminated foods or beverages have caused the limit for arsenic to be placed at 0.05 ppm. Among the more interesting of these observations was the outbreak of chronic arsenic poisoning which took place in the vicinity of Manchester, England, in 1900. Over 500 cases of poisoning were traced to beer containing 2–4 ppm. of arsenic (3).

In establishing safe limits for toxic substances in drinking water, it should be borne in mind that people are always exposed to sources of poison other than water. Thus, vegetables or fruits may contain small amounts of lead, arsenic and fluoride from insecticide residues. The air of the factory in which a man is employed may carry small amounts of lead dust. And the dust or fumes of automobile motors burning "ethyl" gasoline may add to the lead absorption of the individual. Therefore, limits for toxic contaminants in the water supply should be viewed conservatively in the light of other possible sources of absorption.

Other toxic substances which may conceivably be found in drinking water are selenites, cyanides, chromates, iron, aluminum, manganese, sodium, copper, zinc and cadmium. Of these, iron and manganese, because of their color, taste and staining properties, are more of a nuisance than a health hazard. Aluminum, copper and zinc are so little toxic that they hardly warrant consideration. Certainly, in concentrations which do not produce an astringent taste, they are of no toxicological significance. Selenites are rarely found in water supplies in this country, and cyanides could only find their way into water supplies as a result of gross carelessness in disposing of industrial wastes (from electroplating or case-hardening operations) or from deliberate sabotage. Further study is indicated, however, to determine the extent of the hazard of both cyanides and selenium in water supplies.

With increasing knowledge of the toxic effect of sodium in persons suffering from cardio-renal disease, it may very well be found that such patients should not only be placed on a low-salt diet, but should refrain from drinking water which is high in sodium salts. Because the remaining two elements, chromium and cadmium, have recently come into question as possible toxic contaminants of water supplies, it will be worth while to consider the available information pertaining to their hazards.

Chromium Hazards

Chromium may find its way into drinking water in two distinct chemical forms. The trivalent form, such as occurs in chrome-alum or in traces dissolved from stainless steel, is of no toxicological significance (4). The hexavalent or "chromate" stage—chromic acid and its salts, the chromates and bichromates—however, is decidedly poisonous, particularly if

taken in single large doses. The chromates are powerful oxidizing agents and probably owe their toxic properties to this action.

The possibility of chromate contamination of water supplies stems chiefly from two industrial applications: the use of chromic acid electrolyte for chromium plating of metals and the use of chromates or bichromates as corrosion preventives in cooling water systems. The careless disposal of wastes from either of these sources may give rise to appreciable chromate concentrations in sewage or water from adjacent wells (5). Thus far, the principal complaint against chromate contamination has come from sanitary engineers, who have found that even low concentrations in sewage have interfered with normal bacterial action in the activated-sludge process (6).

An instance of this type of water supply contamination occurred at Douglas, Mich., in 1947. A metal products factory in this city had done considerable chrome plating for a period of years, during which the disposal of waste electrolyte had posed a serious problem. It could not be emptied into either Lake Michigan or adjacent streams without endangering fish life and running afoul of the federal government or the Michigan Stream Control Commission. Finally it was dumped into an adjoining abandoned gravel pit. The gravel contained considerable limestone, which, it was thought, would neutralize the acidity of the waste. A thousand feet from this waste pit, however, was a group of six 30-ft. wells from which Douglas obtained the bulk of its water supply. When water from these wells took on a yellow tinge, chemical analysis showed it to contain 11 ppm. of hexavalent chromium, making it necessary to abandon these wells as a source of water supply. In other areas, this chrome waste disposal problem has been met in a satisfactory manner by first reducing the hexavalent chromium to the trivalent state by means of iron turnings, sulfur dioxide or barium sulfide and subsequently treating the reduced chromium with alkali (lime or soda ash) to precipitate chromium hydroxide.

Most human chromate poisoning recorded in the medical literature consists of acute cases in which chromates have been swallowed or chronic cases of local irritation and corrosion of skin and mucous membranes resulting from chromate dusts or the mist which rises from the electrolyte during chrome plating. There is no record of human poisoning where small amounts of chromates have been ingested over a considerable period. It is thus necessary to turn to meager and unsatisfactory reports of animal experiments in order to draw inferences about the probable hazard to man resulting from repeated small doses.

Early experiments on guinea pigs, rabbits and dogs were conducted by Ophüls (7) during the period 1908–12. The primary purpose of these tests was not to determine the chronic toxicity of chromates but rather to study the slowly developing kidney disease which results from chromate administration. Repeated injections of small amounts of potassium bichromate continued over a period of one to eighteen months showed little evidence of poisoning in the animals. In these experiments, the size of dose and the interval between doses varied so greatly that it is difficult to draw any conclusions other than that repeated small doses (4–6 mg. per kilogram of body weight) have little effect.

In 1935 Brard (8) gave potassium bichromate to dogs in daily doses of 1.2–2.0 mg. per kilogram of body weight and found that over a period of three months his animals lost weight and strength, became anemic and finally died.

The best controlled experiments on the chronic toxicity of chromates were conducted by Gross and Heller (9) in 1946. They placed measured percentages of potassium chromate in the drinking water furnished to growing white rats and noted the influence on the growth curve as compared with a control series of animals receiving ordinary water. At concentrations of 500 ppm. potassium chromate in the drinking water, no harmful effects were noted and digestion of food appeared to proceed normally. Higher dosages produced a stunting of growth. It is noteworthy that these observers found no chromium in either the blood or urine of their animals, which suggests that there is little absorption of chromium from the gastrointestinal tract into the blood.

It would appear that such data are hardly adequate to serve as a basis for a safe limit on chromium in water supplies. It is interesting to note that the American Standards Assn. has adopted 1 mg. of chromium trioxide per 10 cu.m. of air as the limit for chrome mist or dust in factory air. The U.S. Public Health Service standard of 0.05 ppm. of hexavalent chromium in drinking water seems amply safe; it may be found too rigid when more exact data become available.

Cadmium Hazards

The question of chronic cadmium toxicity stems from a quite different source. During the metal shortages occasioned by World War II, cadmium plating was used to replace zinc and tin as a rust-resistant coating for steel. Where food containers were "tinned" with cadmium, many cases of acute cadmium poisoning developed. Ice cubes or ices frozen in cadmium-plated trays, ice cream or cottage cheese packed in cadmium-plated cans, or lemonade made in cadmium-plated pails or tubs gave rise to a wave of acute poisonings characterized by vomiting, diarrhea and abdominal cramps. Although alarming and painful, these attacks rarely resulted in any permanent injury. Other poisonings have arisen from the inhalation of cadmium dusts in industry. These, too, are chiefly acute cases. The use of cadmium-plated pipe fittings for water conduits was adopted in many areas without reported difficulty. Nevertheless, because cadmium is known to be poisonous, the question of the possible hazard of chronic cadmium poisoning from this source was referred to the Engineering Div. of the Michigan Health Dept.

In the periodic table of the elements, cadmium is found in the same group as zinc and mercury, lying between them, so that one would expect its toxic properties to resemble those of its two neighboring metals, particularly zinc, since, structurally, the cadmium atom is much closer to zinc than it is to mercury. This inference is borne out by experiment. Cadmium is somewhat more cumulative in its effects than zinc but not nearly so much as is mercury. Cadmium causes some kidney damage, thus resembling mercury to some extent.

Schwartze and Alsberg (10) found that experimental animals died when placed on a diet containing 250 ppm. of cadmium. Cats were able to tolerate daily doses of 100 mg. of cad-

mium fed with 100 g. of meat. There was some evidence of cadmium storage in liver, kidney and spleen. Hessel (11), in similar experiments, confirmed these findings and noted that cadmium was very slowly excreted in the urine and feces and that deposition occurred in liver and kidney tissue.

In rat growth experiments conducted by Johns, Finks and Alsberg (12), varying amounts of cadmium chloride were mixed into the diet. If the concentration of cadmium in the diet exceeded 125 ppm., the animals died after a period of up to five or six months. When the diet concentration was 125 ppm., the animals lived as long as nine months. In similar rat growth tests made by Wilson, deEds and Cox (13), animals on a diet containing 31 ppm. of cadmium showed lower growth curves, while those on a 62-ppm. diet were definitely stunted and became anemic. Duplication of these rat tests by Fitzhugh and Meiller (14) showed that, although young rats receiving a diet containing 45 ppm. cadmium were stunted in growth and anemic, those receiving 15 ppm. were normal. Ginn and Volker (15) made similar experiments but gave some of their rats drinking water containing 50 ppm. of cadmium, while others were given food containing the same concentration. Animals receiving 50 ppm. in food made a normal gain, but those receiving it in water were definitely stunted and anemic.

No chronic toxicity data are available on human subjects, human poisonings being mostly acute. A concentration of 1.0 mg. of cadmium per 10 cu.m. of air has been advocated as a safe limit for factory air, but Hardy and Skinner (16) feel that this figure is too exacting. Some of their employees worked regularly in atmospheres containing eleven times the recommended limit.

With such conflicting, inadequate and unsatisfactory data, it is impossible to form any opinion concerning the toxic limit for cadmium in water supplies (17). Certainly, it is far less dangerous than mercury or arsenic, but correct evaluation of the hazard involved necessitates the acquisition of new experimental evidence in both animals and man. These data should be accompanied by engineering tests to determine the extent of cadmium contamination of water supplies by pipe coatings under varying conditions of temperature, time of contact and water composition.

In the author's opinion, safe limits for toxic contaminants in water should be based upon sound experimental evidence rather than mere conjecture. For tentative standards, it may be justifiable to accept conservative estimates, but these should be maintained only until more accurate experimental data become available.

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Discussion

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As pointed out by the author, the physical effects of several hazardous elements are definitely known and their limits are well established. Little is known, however, of many other elements and much remains to be done. A review of the literature shows a wide variation in the reported amounts of certain elements producing toxic effects. When the effects of certain elements are unknown it is right to set limits, even though they may be arbitrary—in fact, one should bend over backward to be on the safe side. Such limits should not, however, be retained indefinitely, because too strict standards may work an undue economic hardship upon a water producer by forcing the abandonment of one supply and the development, at much greater expense, of a substitute supply with a lower content of supposedly objectionable material. An immediate investigation should be initiated to determine the maximum limits, and revisions should be made as soon as the proper information becomes available.

As an example, within the past few years investigation has shown that copper and zinc are not nearly as harmful as was once thought and their limits have been raised. The limiting value of fluorides has been confined to a very narrow range. The U.S. Public Health Service has not placed any limits on boron, because, as far as now known, it is not harmful to humans. But, on account of its deleterious effect on many plants, particularly the citrus varieties, the U.S. Dept. of Agriculture has suggested limits for the boron content of irrigation water. At one time the limit was 1 ppm. for citrus grove irrigation water, a figure subsequently lowered to 0.5. Now the effect of nitrates, long believed of comparatively little signifi-

cance, is questioned, and it is possible that very definite limits will be set for these substances.

The physiological effects of selenium are still uncertain, and more studies are needed on this element, as well as on chromium and cadmium. In fact, all the elements and many compounds present in water should be thoroughly checked for toxic or physiological effects. Lithium, for example, exploited for years by purveyors of so-called medicinal spring waters, has now been found harmful in quantities used in salt substitute formulas when low-sodium diets are prescribed, so it may well be another element requiring limiting concentrations for optimum health.

Not so long ago an effort was made to produce radioactivity in water through the use of radium compounds. This practice, of course, was stopped, but not before many people had been injured by the radiation products.

A further possibility for investigation is the use of silver as a sterilizing agent. Although perhaps not dangerous from a health standpoint, the continued intake of water containing catadyn silver may produce pigmentation of the skin or other abnormal effects.

It has also been suggested that the formation of certain compounds in water due to treatment with chlorine, and its reaction with other chemicals present, might be objectionable physiologically. The writer is acquainted with a prominent physician who maintains very stoutly that the use of chlorine for water treatment is productive of cancer. An obvious refutation of this argument is the existence of cancer for many years before chlorination was adopted and its presence in areas where chlorination is not generally used. Since negative information is frequently as valuable as positive, however, definite proof that chlorine compounds in water are not harmful might be a very substantial asset from a public relations standpoint.

Since the country's natural water sources are increasingly subject to pollution from industrial wastes of all types, the investigation of the toxic effects of the heavy elements and other compounds would make a good research project for a university or the U.S. Public Health Service. A University of Michigan request for a grant from the U.S. Public Health Service for this purpose was turned down for reasons unknown to the writer. The expenditure of time and money needed to obtain this information would appear to be justified.

Freezing of Water in Exposed Pipelines

By Thomas M. Riddick, Norman L. Lindsay and Antonio Tomassi

A contribution to the Journal by Thomas M. Riddick, Cons. Engr. & Chemist, New York; Norman L. Lindsay and Antonio Tomassi, both with Thomas M. Riddick, Cons. Engr. & Chemist, New York.

IN 1949 the senior author was retained by an industrial firm to make a water works survey for a proposed pulp mill. The installation required the design of a water filtration plant of 30-mgd. capacity with provision for anticipated expansion in the near future to 50 mgd. Raw water was to be drawn at a relatively constant rate from an impounding reservoir created by the construction of a dam, the location and elevation of which were predetermined by topographic conditions.

The mill site was situated about 3½ miles from the dam, and elevations were such as to necessitate absolute minimum head losses in the transmission lines in order to maintain the required pressure at the mill by gravity. Therefore, the rapid sand type of water filtration plant had to be located at the highest level possible, and overall friction losses in the transmission pipelines were limited to the low value of 15 ft.

Raw water was to be conveyed a short distance from the reservoir to the plant site by a single line. Two 48-in. transmission lines were selected to connect the plant to the mill, one to deliver filtered and the other completely treated water. Complete removal of color and turbidity was required for only half the supply, while filtration alone was adequate for the remainder. Cost estimates showed that the interest and amortization of the dual 48-in. lines (as compared with a single 60-in. line) was much more than offset by the savings effected through elimination of one-half the sedimentation basin capacity and reduction by half of the cost of treatment chemicals.

The rock line along the route of the pipe was practically at grade, which precluded placement below ground. Neither was there sufficient fill material economically available for cover. Since the project is located in a cold climate and the long transmission lines were to be exposed, it was necessary to investigate the possibility of freezing in the pipe.

Also, the lines had to be designed for an ultimate capacity of 50 mgd. (25 mgd. each), whereas flow during the first few years of operation would be only 15 mgd. each. This condition further reduced the already low velocity caused by the necessity of limiting head losses in the line, and the low velocities, together with the long length of pipe (16,800 ft.), resulted in a detention period of 2½ hours at the lower

rate of flow. The hazard of freezing was thereby accentuated.

Previous Investigations

Many pipelines or penstocks of large diameter have been installed in cold climates, and little trouble from icing conditions has been reported where lakes or reservoirs are the source of supply. Practically all such lines, however, are characterized by relatively high velocities and short runs, conditions which are inimical to freezing. The satisfactory performance of these installations is probably responsible for the well worn, though erroneous, adage, "Running water doesn't freeze." There has been and still is considerable difficulty due to frazil ice in submerged intakes on rivers subject to prolonged freezeovers, and this phase of icing has been subjected to extensive investigation.

Previous experience with either uncovered penstocks or submerged intakes does not, however, provide a basis for evaluating the design and operating conditions of the pipeline under discussion with regard to ice formation, and no installations comparable in terms of length of line, velocity, detention and temperature conditions can be found in the literature. Communication with several practicing hydroelectric engineers failed to reveal any worth-while precedent or rational method for determining conditions which would result in freezing. Primary investigations, therefore, had to be made, employing fundamental laws of limnology and heat transfer.

The abstract properties of water have been studied over a long period and were competently assembled and reported in 1940 by Dorsey (1) of the National Bureau of Standards. The late Howard T. Barnes (2, 3) of Canada contributed a life's work on the practical aspects of freezing, and his publications, dating from 1908 to 1928, are outstanding. His investigations, however, dealt principally with frazil and anchor ice formed in large flowing rivers such as the St. Lawrence. Whipple (4), Birge (5), Welsch (6) and others have contributed greatly to the field of limnology and have firmly established the principles governing the temperature variations of reservoir and lake waters. It is only by tying together the works of such investigators and numerous others who formulated the fundamental laws of heat transfer that a rational method can be evolved for evaluating the factors controlling the freezing of water in exposed pipelines. The extensive research on rainmaking and icing of airplanes now under way will undoubtedly add substantially to the existing knowledge of ice formation.

Types of Freezing

When the temperature of water falls below 32.00°F., freezing may occur in a number of ways. From the standpoint of freezing in exposed pipelines, two types of ice formation are important: frazil ice, which generally forms en masse at temperatures of 31.90° to 31.99°F.; and the creation of solid ice crystals, which progressively grow in size and may be formed over a wide range of temperatures below 32.00°F. Supercooling, which is today considered a normal rather than unusual occurrence, can produce either type of ice formation. The variance in the findings of a large number of investigators gives evidence of the unpredictability of results obtained when

different (or even the same) types of waters are supercooled.

It is well established in both theory and practice that complete freezing of water can only be accomplished after extracting 144 Btu. of heat per pound of water when the temperature of the mass has been lowered to 32°F. Also, neglecting the phenomena of supercooling, the total pounds of ice formed at any time is directly proportional to the percentage withdrawal of the 144 Btu. required for complete freezing. This factor, together with thermal movement and the insulating effect of thick layers of ice, is largely responsible for the fact that the sheet ice covering a reservoir, lake or river is seldom more than a few feet in thickness, even in very cold climates.

Before developing a formula to evaluate the conditions which can produce dangerous icing in exposed pipelines, it is advisable to consider further the types of freezing which can occur and to establish whether or not *any* ice formation can be safely tolerated. It is also well to bear in mind the distinction between so-called "pure water" and that collected in quantity from natural watersheds, with or without treatment. Reasonably pure water can be obtained only under the most carefully controlled laboratory conditions. It is necessary to precipitate the entrained dust with flocculent materials such as barium hydroxide and to employ repeated distillations *in vacuo*. Even then, the small percentage of dissolved gases may be partially responsible for the variations encountered in carefully controlled supercooling experiments. At the General Electric Research Laboratory in Schenectady, N.Y., Smith-Johannsen (7) demonstrated that "pure water" normally freezes at a temperature of about −20°C. (−4°F.) rather than 0°C. (32°F.), unless the water is in contact with ice crystals.

Natural water, with or without treatment by coagulation and filtration, generally contains calcium and magnesium bicarbonates, sodium and potassium sulfates and nitrates, sodium chloride, silica, the dissolved gases of oxygen, nitrogen and carbon dioxide, and some organic matter, silt and algae.

The turbidity due to microscopic particles of silt and algae, even in a filtered water, is generally sufficient—at least in volume, if not in composition—to serve as nuclei for ice crystal formation, which precludes supercooling to the degree possible with "pure water." Even raw, unfiltered waters, however, may be supercooled, under conditions of agitation and in vessels with relatively "rough" walls, to as low as 3° or 4°F. below the freezing point, although the temperature at which crystal formation first occurs often seems unpredictable and uncontrollable. This observation is confirmed by laboratory experiments with raw New York City tap water run in connection with this pipe freezing problem.

If ice formation in exposed pipelines could always be relied upon to begin at a fraction of a degree below 32°F. (no appreciable supercooling), if this ice plated out in discrete and growing crystals on the pipe walls, and if these crystals grew inward without exerting any tangential pressure on the pipe walls, then a reasonable amount of ice formation could be safely tolerated. In fact, as will be subsequently shown, some insulating effect would thereby be obtained from the ice itself. Since

the shattering stress of ice is approximately 400 psi., even if crystals grew so as to exert full tangential stress on pipe walls, no danger would normally be encountered until more than ¼-in. thickness of ice had developed. If *no* stress were exerted tangentially, although this seems improbable, a heavy coating of ice could provide a very appreciable insulating effect, and the increase in velocity of the flowing water due to the decrease in effective pipe size would reduce the detention period and protect most lines during severe and attenuated periods of cold weather.

Although the conditions mentioned above are possible, they are the least probable. The most likely method of freezing is the formation of frazil ice, which normally occurs simultaneously in a large mass of water upon supercooling from 0.01° to 0.10°F. The size of these ice particles varies from "microscopic" to about 0.1 mm. in diameter. Because of their small size, they tend to remain uniformly dispersed throughout the mass rather than rising to the surface.

These crystals become readily attached to any roughened surface and would start to build up at a valve, pipe coupling, bend or welded seam. Additional crystals of frazil ice brought down by the flowing water would then rapidly enlarge this mass and the pipeline would very soon become entirely blocked.

This condition is common in submerged intake lines handling river water at a temperature of 32.00°F. (minus 0.01° to 0.10°F.), but bursting pressures are seldom achieved, because the ice remains in a slushy or coarsely divided state. Such freezing in a pipe exposed to low air temperatures would probably be disastrous, as it would be quite difficult to ascertain the point of stoppage, and the line would probably freeze solid and burst before clearance could be effected.

The author has experienced one such instance which resulted in the bursting of a 16-in. cast-iron main laid above grade. The water in question was drawn from a reservoir at a temperature of 37°F., and an attempt was made to discharge it through cast-iron aerator piping which had been out of service and exposed to subzero temperatures during the previous night. Although the 16-in. control valve was opened manually as quickly as possible, slush or frazil ice formed almost instantaneously in the aerator line and blocked it off completely. Bursting occurred about four hours later.

A third and likely method of freezing, which was clearly demonstrated by laboratory experiments with raw water, is the formation of discrete needles and plates of relatively large size after an unpredictable period of supercooling. Although the water used in these experiments contained ample turbidity and algae to serve as nuclei for crystal formation, and the sample (in a cylindrical copper container) was continuously agitated by a mechanical stirrer, supercooling continued for 3° or 4°F., at which point of heat extraction a mass of needlelike crystals formed simultaneously and the temperature rose from 28° to 32.00°F. Ice crystals remained in suspension during the rest of the freeze, with further cooling resulting in the gradual buildup of solid sheet ice on the cylindrical walls of the vessel. But, translated into terms of flow in an exposed pipeline, the effect of these crystals would be to produce

a slurry-like mass which would retard the rate of flow of water in the pipe to such an extent that it could not be readily emptied, and clogging of the pipe would probably occur.

A consideration of these methods of freezing indicates that *no* ice formation can be safely tolerated. Therefore, the temperature of water at the discharge end of the pipeline should always be maintained above 32.00°F., and, from a practical standpoint, the safety factor should, if possible, be 0.5°F., although 0.1°F. would actually prevent ice formation.

Limnological Considerations

There is only one practical method of preventing the freezing of water in

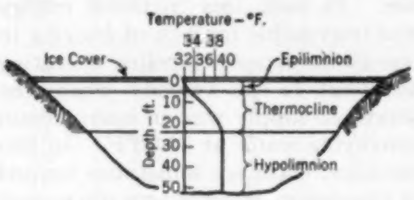

FIG. 1. Thermal Stratification

pipelines which have long detention periods and are exposed to cold weather: to withdraw "warm" water from the reservoir or lake supply. By warm water is meant water above the freezing point and with a Btu. content sufficient to take care of the heat losses which occur along the transmission line.

The science of limnology is well developed as far as temperatures are concerned. Stratification, due to changes in the density of water with changes in temperature, causes bottom waters of deep lakes and reservoirs to approximate closely 4°C. (39°F.). After the freezeover the waters in the upper stratum or epilimnion will closely approximate a temperature of 32°F. This zone, however, is usually only a few feet in depth during the winter months. The temperature of waters from the lower zone of the thermocline can be safely predicted at 3°C. (37°F.) and that of waters in the bottom zone, at 4°C. (39°F.), as indicated in Fig. 1, which shows thermal stratification in North Temperate and Arctic Zone lakes in winter. Although these conditions of temperature are basic, they apply primarily to reservoirs and lakes not subject to heavy withdrawal of water or to considerable fluctuations in water levels.

Unlike well stratified waters, the temperature of the feeder streams, during periods when air temperatures average 10° to 20°F. below freezing, will generally be 32.00°F. ± 0.1, and such streams will often carry frazil ice. Because precipitation is mainly retained as ice, snow or frost on the watershed, runoff during the winter months is generally light. If a main feeder stream, however, discharges into a reservoir close to the gatehouse or outlet, the stratification value of 38°–39°F. should be lowered to allow for the short-circuiting of colder water in the area adjacent to the mouth of the stream. This can be a very important consideration in the proper location of reservoir outlets or gatehouses.

Where it is necessary to employ spray aeration at a water treatment plant during the winter months, a further lowering of temperature is to be anticipated. Therefore, stream waters can seldom be aerated with spray nozzles if air temperatures are far below freezing, since frazil or slush ice will form copiously in the aerating basin.

Heat Input

The heat available to prevent ice formation in exposed pipelines is almost entirely due to the degrees of temperature of water above 32.00°F. This heat credit can be mathematically expressed in terms of Btu. per square foot of exposed pipeline per hour:

$$H_1 = \frac{\text{(Flow of water, lb. per hr.) (Temp. at inlet end, °F.} - 32)}{\text{Area of exposed pipeline, sq.ft.}}$$

This is equivalent to:

$$H_1 = \frac{1{,}325Q(T_w - 32)}{Dl} \quad \text{.. Eq.1}$$

in which H_1 is the Btu. per square foot per hour available from the specific heat of the water; Q is rate of flow, in million gallons per day; T_w is the temperature of the water, in degrees Fahrenheit, at the inlet end of the pipeline; l is the length of the pipeline, in thousands of feet; and D is the internal diameter of the pipeline, in inches.

Since minimum temperatures generally occur at night, and cold periods are often characterized by cloudy weather, no radiant-heat gain from sunshine or ground surface can be relied upon. Otherwise, this energy might be appreciable, as is evidenced by promising research now being undertaken in connection with solar heating of houses.

The only other credit (besides applied heat) which can be considered as adding to the heat available to prevent ice formation is the frictional heat generated by the passage of water through the pipeline. This can be expressed as:

$$H_2 = \frac{\text{(Flow of water, lb. per hr.) (Total friction loss in pipeline, ft.)} \times 0.0154}{\text{Area of exposed pipeline, sq.ft.}}$$

This is equivalent to:

$$H_2 = \frac{1.70Qf}{D} \quad \text{......Eq. 2}$$

in which H_2 is the Btu. per square foot of pipe per hour generated by frictional energy; f is the friction loss, in feet, per 1,000 ft. of pipe, determined from the Hazen-Williams or some other appropriate formula; and D is the diameter of the pipeline, in inches.

This heat gain is negligible for large-diameter pipe at low velocities but may be quite appreciable for small-diameter lines operating at high velocities. In fact, this frictional energy was responsible for lack of freezing in one 20-in. exposed pipeline operating at about 15-fps. velocity where the source of supply was an open stream conveying water at 32.00°F. In this instance, frazil ice sometimes formed in the stream, but was actually melted in passing through the pipeline.

The sum of H_1 and H_2 represents the total heat expendable in any gravity pipeline to prevent ice formation.

Heat Losses

When a pipeline carrying water is surrounded by colder air, a flow of heat from the water to the atmosphere must occur. There are three barriers to this transfer of heat, and four if the pipe is insulated.

Water Film

The first barrier is the water film. The coefficient, h_f, represents the Btu. per square foot per hour transmitted

through the thin, slow-moving film of water located around the internal circumference of the pipe wall. A simplified equation for this condition (8), applying to water close to the freezing temperature, is:

$$h_f = \frac{202v^{0.8}}{D^{0.2}} \quad \text{.......Eq. 3}$$

in which h_f is the Btu. per square foot per hour per degree Fahrenheit differential transmitted from the bulk of the water at a temperature of 32°F.

TABLE 1

Values of D and v

D Pipe Diam. *in.*	$D^{0.2}$	v Avg. Water Velocity *fps.*	$v^{0.8}$
6	1.4	1	1.0
12	1.6	2	1.7
18	1.8	3	2.4
24	1.9	4	3.0
36	2.0	5	3.6
48	2.1	6	4.2
60	2.2	8	5.3
72	2.3	10	6.3
84	2.4	12	7.3
96	2.5	14	8.3
		16	9.2
		18	10.1
		20	11.0

through the film of the pipe wall; v is the average velocity of water in the pipeline, in feet per second; and D is the diameter of the pipe, in inches. For convenience, various values of v and D to the required powers are given in Table 1.

It is evident from Eq. 3 that the transfer of heat through the water film is extremely high even with large-diameter pipe at low velocities and increases greatly with small-diameter pipe at high velocities.

Conductivity

The second resistance to the flow of heat from water to the air is that offered by the pipe material. This depends on the conductivity of the material, the thickness of the pipe wall and the temperature differential between the inside and the outside walls of the pipe. Fourier's law expresses this relationship thus:

$$q = \frac{k\Delta t}{L} \quad \text{........Eq. 4}$$

TABLE 2

Conduction Heat-Transfer Values

Substance	k*	Assumed Thickness *in.*	Heat Transfer Value *Btu./sq.ft./hr./°F.*
Pipe material			
Steel	420	0.25	1,240
Cast iron	385	0.75	515
Concrete	5.3	5.0	1.1
Wood stave	1.0	2.0	0.5
Aluminum	1,410	0.25	5,640
Asbestos cement	4.5	1.0	4.5
Insulator			
Dry air	0.17	2	0.08
Water	4.0	2	2.0
Ice	15.6	2	7.8
85% magnesia	0.4	2	0.2
"Foamglas" †	0.4	2	0.2

* Btu. per square foot per hour per degree Fahrenheit differential per inch thickness of material.

† A product of Pittsburgh Corning Corp., Pittsburgh, Pa.

in which q is the Btu. transmitted by conduction through the pipe wall per square foot of pipe surface per hour; k is the thermal conductivity of the pipe wall, in Btu. per square foot per hour per degree Fahrenheit per inch thickness of the pipe wall; Δt is the temperature differential, in degrees Fahrenheit (32 minus air temperature, for freezing conditions); and L is the thickness of the pipe wall, in inches.

Therefore, $\frac{k}{L}$ is the rate of heat transmission by conduction per degree Fahrenheit differential through a pipe wall; $\frac{k'}{L'}$ is the corresponding factor for heat conduction through pipe insulating materials, if employed. Values for heat transfer by conduction, for pertinent materials, are given in Table 2.

Radiation and Convection

The third and fourth means of liberating heat (from the external pipe walls to the air) are radiation and convection, and the resistive value of these combined losses is referred to as surface resistance. Radiation is the transmission of energy by "heat waves" from the surface of one body to another. Convection is the transmission of heat within a fluid (such as air or water) and, under normal conditions, is caused by changes in the density of the receiving fluid. The relative motion within the fluid may also be produced by mechanical means, such as the agitation of the fluid during heating, or by continued displacement of air (when this is the fluid) due to wind movement.

Both radiation and convection have been subjected to considerable study, and many theoretical and empirical equations have been evolved for their evaluation. The nature of these heat losses, however, often causes widely divergent results between theoretical and actual heat transfers when operating conditions are not comparable with those from which the formula was derived. It is for this reason that radiation and convection are often combined in such computations and that preference is given to empirical data.

The fundamental law of radiation is that expressed by the Stefan-Boltzmann equation, which, in engineering units, may be stated as:

$$H_r = 0.17E\left[\left(\frac{T_1}{100}\right)^4 - \left(\frac{T_2}{100}\right)^4\right] \quad \text{..Eq. 5}$$

in which H_r is the Btu. per square foot of radiating surface per hour; E is the emissivity coefficient, ranging from 0 (complete reflection) to 1 (complete absorption); and T_1 and T_2 are the absolute temperatures of the radiating bodies, in degrees Fahrenheit.

TABLE 3

Emissivity Factors

Material	Emissivity Factor E
Asphaltic paint (black)	0.9
White enamel	0.9
Aluminum paint	0.4
Cast iron	0.7
Wood (dressed)	0.9
Asbestos cement	0.9
Aluminum	0.1
Brass or copper (with patina)	0.5

In this problem, $T_1 = 492°$ for water and $T_2 = 455°$ for air, and Eq. 5 for unit temperature differential (but for full temperature differential conditions) becomes:

$$\frac{H_r}{\Delta t} = h_r = 0.17E\,\frac{\left(\frac{492}{100}\right)^4 - \left(\frac{455}{100}\right)^4}{37}$$

$$= 0.77E \text{ Btu./sq.ft./hr./°F. differential} \quad \text{.... Eq. 6}$$

Factors of emissivity for computing radiation from the various pipe metals and coatings at a temperature of about 32°F. are approximately as given in Table 3.

Convection losses are a function of the temperature differential and the size of the pipeline. A simplified equa-

tion applying to horizontal pipe in still air is given by McAdams (8):

$$h_c = 0.55\left(\frac{\Delta t}{D}\right)^{0.25} \quad \ldots\ldots \text{Eq. 7}$$

in which h_c is the Btu. transmitted by convection per square foot per hour per degree Fahrenheit differential; Δt is the temperature differential between the outside pipe wall and the air; and D is the diameter of the pipeline, in inches.

Convection losses increase very appreciably with wind velocity, and

TABLE 4

Wind Velocity Factors

Wind Velocity *mph.*	Factor $\sqrt{\frac{V+0.8}{0.8}}$
5	2.7
10	3.7
20	5.1
30	6.2
40	7.1

Langmuir (9) gives the following formula to express this relation:

$$h_{cv} = h_c\sqrt{\frac{V+0.8}{0.8}} \quad \ldots\ldots \text{Eq. 8}$$

in which h_{cv} represents the convection losses (as above), corrected for wind velocity, and V is the wind velocity, in miles per hour. The factor by which convection losses in still air must be multiplied for different wind velocities is given in Table 4.

Both radiation and convection formulas can be solved when the temperature of the outer pipe surface during balanced conditions of heat exchange is known. When thin-walled and very highly conductive pipe materials such as steel are employed, it is reasonable to assume that this temperature (T_1) approximates the temperature of the water in the pipeline, the lower limit of which is 32.00°F. (or 492°F. absolute).

When pipe walls are thick and relatively nonconductive (as for concrete), the exterior surface will be closer to the temperature of the surrounding air. Resistance to conduction rather than to convection and radiation will then be the main factor governing heat transfer. Under such conditions, radiation and convection losses are smaller than those given by Eq. 6 and 7, which are based upon the full temperature differential. Divergency, however, is on the side of safety and is not appreciable, since the range of temperature for this problem is restricted. Exact values could be determined by a trial-and-error procedure, but this high degree of accuracy is unwarranted.

Application of Principles

The consideration of each significant step of heat gain and heat loss in exposed pipelines now enables the inclusion of all of them in a single formula to express each of these quantities. The various factors are recapitulated in Table 5, together with a listing of terminology and required data, and are shown diagrammatically in Fig. 2.

The heat available to prevent freezing is:

$$H_{\text{input}} = \frac{Q}{D}\left[\frac{1{,}325(T_w - 32)}{l} + 1.7f\right] \ldots \text{Eq. 9}$$

The heat losses can be expressed as:

$$H_{\text{loss}} = \frac{\Delta t}{\frac{1}{h_f} + \frac{L}{k} + \frac{L'}{k'} + \frac{1}{h_r + h_{cv}}} \ldots \text{Eq. 10}$$

TABLE 5

Recapitulation of Heat Balance Factors

Terminology and Required Data

Symbol	Meaning	Unit	Symbol	Meaning	Unit
	Climatological			Pipeline	
T_w	water temp. at pipe inlet	°F.	v	velocity	fps.
			C	friction coefficient‡	
T_1	water freezing temp.	°F. abs. = 492	f	friction loss‡	ft./1,000 ft.
T_2	air temp.	°F. abs.*	l	length	1,000 ft.
Δt	temp. differential between air and water ($T_1 - T_2$)	°F.	L	wall thickness	in.
			L'	insulation thickness	in.
V	wind velocity	mph.	k	pipe wall thermal conductivity coefficient	Btu./sq.ft./hr./°F./in.
	Pipeline		k'	insulation thermal conductivity coefficient	Btu./sq.ft./hr./°F./in.
Q	flow rate	mgd.	E	emissivity (pipe wall, insulation)	
D	diameter†	in.			

Heat-Loss Factors

Factor	Heat Transfer§
$h_r = \frac{0.17E}{\Delta t}\left[\left(\frac{T_1}{100}\right)^4 - \left(\frac{T_2}{100}\right)^4\right]$	by radiation
$h_{cv} = \sqrt{\frac{V + 0.8}{0.8}}\left[0.55\left(\frac{\Delta t}{D}\right)^{0.25}\right]$	by convection
$\frac{k'}{L'}$	by conduction (through insulation)
$\frac{k}{L}$	by conduction (through pipe wall)
$h_f = \frac{202v^{0.8}}{D^{0.2}}$	through water film

Heat-Input Factors

Factor	Description
$H_1 = \frac{1{,}325Q(T_w - 32)}{Dl}$	heat available due to temp. of water above 32°F.
$H_2 = \frac{1.70Qf}{D}$	heat generated by friction

* Inside diameter for thin walls; average diameter for thick walls.
† Hazen-Williams formula.
‡ Equals 460 plus air temperature in degrees Fahrenheit.
§ Per degree Fahrenheit differential.

in which the term $\frac{L'}{k'}$ is the resistance of insulating material (if used) of thickness L' and thermal conductivity coefficient k'. It will be noted that convection and radiation are expressed as a joint unit, which then combines in series with the other resistances.

To prevent freezing, the heat input must be greater than or equal to the heat losses, and a factor of safety is highly desirable.

In the pipeline under discussion, the pertinent data are as follows: $Q = 25$ mgd.; $T_w = 35°F.$; $D = 48$ in.; $T_{air} = -5°F.$; $v = 3.1$ fps.; $f = 0.60$ ft./1,000 ft. (at $C = 130$); $l = 16.8$; $\Delta t = 37°F.$; and $V = 35$ mph. The pipe materials considered were steel, wood stave, concrete and aluminum. The heat input in all instances was:

$$H_{input} = \frac{25}{48}\left[\frac{1{,}325(35-32)}{16.8} + (1.7)(0.6)\right]$$
$$= 0.52\,[237 + 1.0]$$
$$= 124 \text{ Btu./sq.ft./hr.}$$

It is evident that, in this pipeline, the gain in heat due to frictional resistance was negligible.

Heat-loss coefficients for film resistance and convection from thin-walled metals are independent of pipe materials, hence common for all instances. These values are:

$$\frac{1}{h_f} = \frac{1}{\dfrac{202(3.1)^{0.8}}{48^{0.2}}} = \frac{1}{\dfrac{500}{2.2}} = 0.0044$$

$$h_{cv} = 6.6h_c = 6.6 \times 0.55\left(\frac{37}{48}\right)^{0.25}$$
$$= 6.6 \times 0.55 \times 0.94 = 3.4$$

Computations of conduction and radiation for $\frac{1}{4}$-in. steel pipe, where $k = 420$ and emissivity $(E) = 0.7$, are as follows:

$$\frac{L}{k} = \frac{0.25}{420} = 0.0006$$
$$h_r = 0.7 \times 0.77 = 0.5$$

Radiation and convection losses are then combined:

$$h_{cv} + h_r = 3.4 + 0.5 = 3.9$$
$$\frac{1}{h_{cv} + h_r} = \frac{1}{3.9} = 0.26$$

The combined overall heat losses are therefore:

$$H_{loss} = \frac{37}{0.0044 + 0.0006 + 0.26} = \frac{37}{0.26}$$
$$= 142 \text{ Btu./sq.ft./hr.}$$

These computations indicate that this 48-in. steel pipeline, at a rate of flow of 25 mgd., would have no safety factor against freezing near its end, as the available Btu. amounts to 124 per square foot per hour, against heat losses of 142. Under operating conditions of 15 instead of 25 mgd. during the first few years of service, the available Btu. would be reduced to approximately $\frac{15}{25}$ of 124, or 75 Btu. per square foot per hour. This, of course, means that the freezing point would be reached at about midlength of the line.

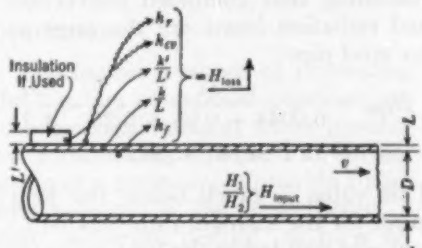

FIG. 2. Heat Balance in Exposed Pipelines

It is also evident, from an analysis of resistances to the various heat losses,

that water film and steel pipe material afford no appreciable retardation to the flow of heat, and that convection and radiation are the sole controlling factors. This result substantiates the assumption that, with thin-walled metal pipe, the exterior wall temperature would approximate the water temperature, thus obtaining the full temperature differential of 37°F.

A study of the formula for heat loss shows that conduction resistance is the only factor subject to wide variations and depends upon the thickness and thermal conductivity of the material employed. For a 5-in. concrete pipeline, for instance:

$$k = 5.3; \frac{L}{k} = \frac{5}{5.3} = 0.95$$

Assuming that combined convection and radiation losses are the same as for steel pipe:

$$H_{\text{loss}} = \frac{37}{0.0044 + 0.95 + 0.26} = \frac{37}{1.21}$$
$$= 30 \text{ Btu./sq.ft./hr.}$$

This value is much below the heat input for the 25-mgd. rate (124 Btu.) and also well below the heat input for the 15-mgd. rate (75 Btu.). Concrete pipelines would, therefore, provide a substantial factor of safety. The same is true for 2-in. wood-stave pipe, where the heat losses would be:

$$k = 1.0; \frac{L}{k} = \frac{2}{1} = 2$$

$$H_{\text{loss}} = \frac{37}{0.0044 + 2.0 + 0.26}$$
$$= 16 \text{ Btu./sq.ft./hr.}$$

The widespread use of wood-stave pipe is no doubt responsible for the absence of operating difficulties due to freezing in numerous long and exposed pipelines located in cold climates. It is probably true, however, that many such lines operate on a very narrow margin of safety, and one notable example of freezing in an exposed 8-mile section of 28-in. wood-stave pipe has been recorded (10).

The formula also seems to offer an explanation for an apparently incongruous condition noted in Canada. In this instance, water at a temperature of about 39°F. during the winter months was discharged from the bottom of a lake to an open stream. After a short traverse of the stream bed it was diverted to a covered wood flume about 1 mile long, where the water temperatures sometimes decreased to 32°F. or below, with the formation of frazil ice. From the flume, the water passed through a mile of 20-in. steel pipe laid at a very steep slope, where the flow velocity was extremely high. Operating data during critical periods were approximately as follows: $Q = 20$; $v = 15$ fps.; f (at $C = 100$) $= 45$; $T_w = 32.0$; and $l = 5$. The heat available to prevent freezing of the line was, therefore, due only to that generated by friction, or

$$H_{\text{input}} = \frac{20}{20}\left[\frac{1{,}325(32-32)}{5} + 1.7 \times 45\right]$$
$$= 1[0 + 77]$$
$$= 77 \text{ Btu./sq.ft./hr.}$$

Since there was no further freezing in the line and sometimes a melting of frazil ice, it seems evident that the heat gain due to friction exceeded the combined heat losses.

Temperatures

In the problem of the freezing of the dual 48-in. pipelines, temperature records were available for a period of 38 years. Since the weather station was situated at a lower and warmer

location, about 5 miles distant from the pipelines, an allowance of a 5°F. drop in temperature was made. Winter months with a mean temperature below 32°F. occurred in 32 of the 38 years of record. Monthly minimum temperatures averaged 10°F. for two months of record and 15°–20°F. for sixteen months of record.

Minimum temperatures ranging from 0 to −5°F. and lasting for several days occurred about 25 times during the 38 years. A characteristic example is given in Table 6. Since

TABLE 6

Low-Temperature Conditions

Date (1947)	Max. Temp. °F.	Min. Temp. °F.
Jan. 28	8	−4
29	0	−7
30	2	−9
31	7	−8
Feb. 1	9	−2
2	20	−2
Avg.	8	−5

a drop in water temperature of 1°F. (from 33.00 to 32.00°F.) can occur in approximately 30 minutes in the steel pipeline at a 25-mgd. rate, and since supercooling to 31.90°F., with the attendant danger of forming frazil ice and clogging, could then take place in three minutes, it is evident that even a one-day period would be ample to produce hazardous conditions if heat losses greatly exceeded heat input.

When pipelines are located in a heavily timbered area and where periods of cold weather prevail only for a short period, it is advisable to measure air temperatures along the route of the line rather than to accept values obtained from a weather station in the general vicinity, since the heat released by forests during cold snaps can be appreciable. When snow covers pipelines during the winter months, the insulating effect of this air cell blanket effectively reduces heat losses.

Well water temperatures are about 50°F. in the northern sections of the United States where wells are the source of supply. This high value should furnish ample heat for the prevention of freezing, even at very low temperatures.

Another point to be emphasized is the advisability of constructing intakes or gatehouses in the deep sections of reservoirs and installing inlets at frequent depth intervals to enable the withdrawal of bottom water at a temperature of 38°–39°F. during the winter months.

Conclusions

The logical methods of preventing the freezing of exposed pipelines are to secure the warmest water possible by withdrawing it from the bottom strata of a reservoir, to provide a high flow velocity in the pipe (hence, a short detention period), to blow off water to waste at the end of the line and to check heat losses by means of insulation.

When waters are untreated, blowoff is readily possible. If a treatment plant is employed, however, its capacity may limit such procedure. Insulation is possible and not prohibitive in price, and the use of such materials as wood or "Foamglas"* (a highly porous glass) seems economically feasible. If the line can be enclosed in any type of cover so as to produce several inches of dead air space around the pipe wall, freezing would be prac-

* A product of Pittsburgh Corning Corp., Pittsburgh, Pa.

tically impossible, even under severe conditions.

It is believed that this analysis of freezing conditions in pipelines offers a rational and easily applied method for evaluating the problem, and that it will be of interest to designers and operators of exposed pipes of large diameter. It should also enable considerable economy to be effected by the location aboveground of many smaller-size transmission mains. It is hoped that this paper will promote further research on the problem, and the receipt of any factual data from operating engineers on the performance of pipelines exposed to cold climates will be appreciated by the authors.

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Erratum

The paper "Design Standards for Large-Diameter Steel Water Pipe" by Walter H. Cates, which was published in the September 1950 Journal (Vol. 42, page 860), contains an error. In line 12, column 1, page 861, the reference to the Barnard article on steel water pipe design is assigned the key number 1. This number was inadvertently duplicated, however, in the second line from the bottom of column 2, page 861, and the reference to the Barnard paper was omitted from the list on page 885. The Barnard reference should have been listed as follows: Barnard, Russell E. Design Standards for Steel Water Pipe. Jour. A.W.W.A., **40**:24 (Jan. 1948)."

Design of Prestressed Concrete Cylinder Pipe

By Hugh F. Kennison

A paper presented on May 24, 1950, at the Annual Conference, Philadelphia, by Hugh F. Kennison, Chief Engr., Lock Joint Pipe Co., East Orange, N.J.

PRESTRESSED concrete cylinder pipe was first commercially manufactured in 1942. Since that time over 500 miles of it have been installed, in sizes from 12 to 69 in. and for operating pressures up to 300 psi. In this paper, the derivations of the design formulas used for prestressed concrete pipe are presented and the backload and beam strengths summarized. Methods are introduced for analysis of pipe subjected to combined internal and heavier-than-normal external loads.

The permissible working stress of steel in conventional reinforced concrete is limited by the low tensile properties of concrete. This limitation necessitates the use of large amounts of reinforcing steel so that deformation at working loads will not exceed the allowable elongation strain of the concrete. By prestressing, the induced tensile load of the steel reinforcement is resisted by an equal compressive load in the concrete core. Under working conditions, this induced compression must be relieved before the concrete can go into tension and reach its allowable elongation. Thus, prestressing greatly increases the permissible deformation of the concrete and allows the use of a relatively small amount of reinforcing steel, working at a high stress.

Historical Background

The first attempt at prestressing concrete was made in Germany in 1888. Experiments at that time failed because the mortar concrete used had insufficient strength to develop an adequate bond between the concrete and the steel. Twenty years later moderate success was achieved in prestressing concrete beams by anchoring the mild-steel reinforcement to end plates, insuring against slippage due to bond failure. Prestressed concrete design improved tremendously in 1927, when high-strength wire and exceptionally strong concrete were first used in France. Since then prestressing has been successfully applied to piles, railroad ties, poles, floor slabs, arches, beams and many other reinforced concrete structures. The development of prestressed concrete cylinder pipe was started in 1937. After five years of extensive research, pipe of this kind was first used for a major installation. Since that time over 3,000,000 ft. has been installed throughout North and South America for all operating pressures and laying conditions commonly encountered in water transmission lines.

Description of Pipe

Prestressed concrete cylinder pipe, illustrated diagrammatically in Fig. 1,

consists of a continuously welded thin steel cylinder with steel bell and spigot rings welded to its ends. The completed steel cylinder, with joint rings attached, is tested hydrostatically for watertightness to about 25,000-psi. tensile stress in the steel. The steel cylinder is then lined with concrete of suitable thickness. After proper curing, the lined cylinder is helically wrapped, at uniform spacing, with high-tensile steel wire under a constant tension. A mortar coating is deposited over the wire and cylinder to protect the steel from corrosion, and, after a curing period, the pipe is ready for delivery to the ditch. A rubber gasket is employed as the sealing element between adjacent pipes. The pipe can be manufactured at either temporary or permanent plants. Generally speaking, pipes for small jobs are made at permanent plants and shipped by truck or rail to the point of installation. On larger jobs, where the saving in freight is substantial, temporary plants can be justified economically and generally are located adjacent to the project.

Pipe Design

The design of prestressed concrete pipe utilizes a continuous, watertight steel cylinder surrounding the concrete core and compressed with it. By prestressing with high-tensile wire to such an extent that the elastic limits of the steel cylinder and the wire subsequently will be reached simultaneously, a most economical design results. The hydrostatic pressure at which this condition of stress in the steel structure occurs is called the elastic-limit pressure. One of the restrictions on design is that the elastic-limit pressure must be at least 2¼ times the normal operating pressure. A second limitation placed on design is that the operating pressure must not exceed the zero-compression pressure—that is, the pressure at which the induced compression in the concrete and steel cylinder core will be reduced to zero. Bursting, elastic and zero-compression pressures can be predicted and substantiated by tests to an accuracy within a few per cent.

At zero internal pressure, the total compressive load of the concrete core

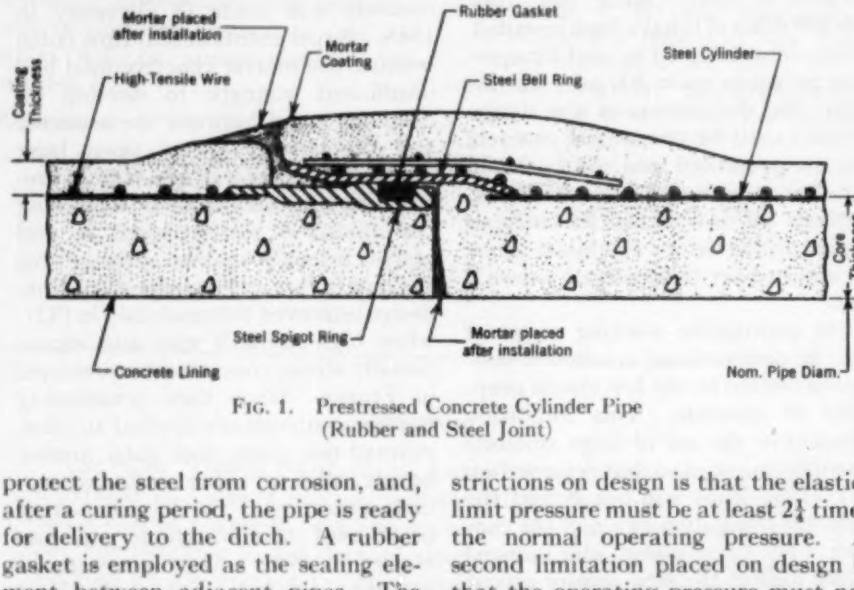


Fig. 1. Prestressed Concrete Cylinder Pipe
(Rubber and Steel Joint)

and steel cylinder equals the tensile load of the high-tensile wire. At the zero-compression pressure, the compression stress in the concrete and steel cylinder is reduced to zero and the high-tensile wire is resisting the entire internal hydrostatic pressure. The increased elongation of the steel from zero pressure to zero-compression pressure is slight, however, since the forces producing tensile stress in the wire are gradually transferred from the core to the water as the hydrostatic pressure is increased.

The behavior of reinforced concrete and prestressed concrete cylinder pipe can be compared by plotting the theoretical and actual conditions of pressure and strain. Figures 2 and 3 illustrate typical pressure-strain gage readings for these pipes. Each curve represents strain gage readings for a 42-in. pipe, reinforced for an operating pressure of 100 psi. and tested hydrostatically. The 42-in. reinforced concrete cylinder pipe is designed at a conventional steel stress of 12,500 psi., the cylinder area is 1.255, the reinforcement area 0.885 and the total steel area 2.140 sq.in. per linear foot. The prestressed concrete cylinder pipe is designed consistent with A.W.W.A. specifications (1); the cylinder area is 0.718, the area of the high-tensile wire 0.408 and the total steel area 1.126 sq.in. per linear foot. The solid lines in Fig. 2 and 3 indicate the theoretical conditions, while the broken lines are typical pressure strain gage curves. The actual strain is less than the theoretical because of the disregard of the tensile strength of the concrete. The strain, X, in Fig. 2 indicates the actual elongation of the pipe at the

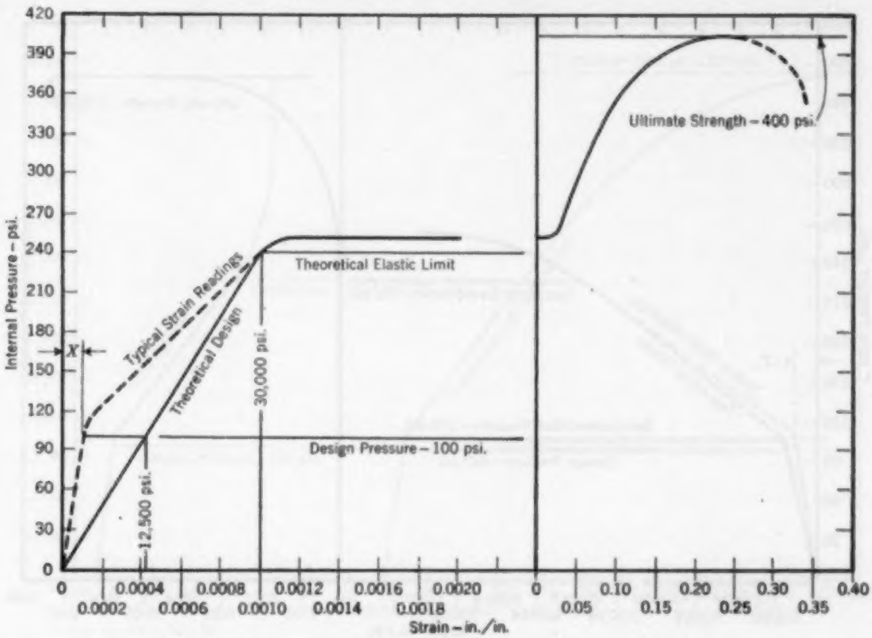


FIG. 2. Pressure-Strain Curve (Reinforced Concrete Pipe)

operating pressure, or 12,500-psi. theoretical stress. It will be observed that, even though the total steel in the prestressed pipe is approximately one-half of that in the reinforced concrete pipe, the actual elongation (X and X_1) of the two pipes at the operating pressure is approximately the same.

Since the cylinder and reinforcement of the reinforced concrete cylinder pipe are at zero stress at zero hydrostatic pressure, Fig. 2 is also a typical pressure-stress curve for this pipe, particularly for stresses below the elastic limit of the steel, the stress being directly proportional to the strain indicated.

Figure 4 is a typical pressure-stress curve of the 42-in. prestressed pipe in Fig. 3. The shapes of the curves for the steel stresses in the cylinder and wire are identical below the elastic-limit pressure and correspond to the strain readings in Fig. 3. Their location, however, is shifted to correspond with the stresses originally induced in the pipe. The concrete compressive stress is not shown but is proportional to the compressive stress in the steel cylinder, in the ratio of their moduli of elasticity. Above the elastic-limit pressure, the strain in the cylinder is controlled by the high-tensile wire. Consequently, the stress in the more ductile cylinder behaves as shown. When the pressure is raised to bursting, the wire fails in pure tension and the cylinder is free to elongate and burst.

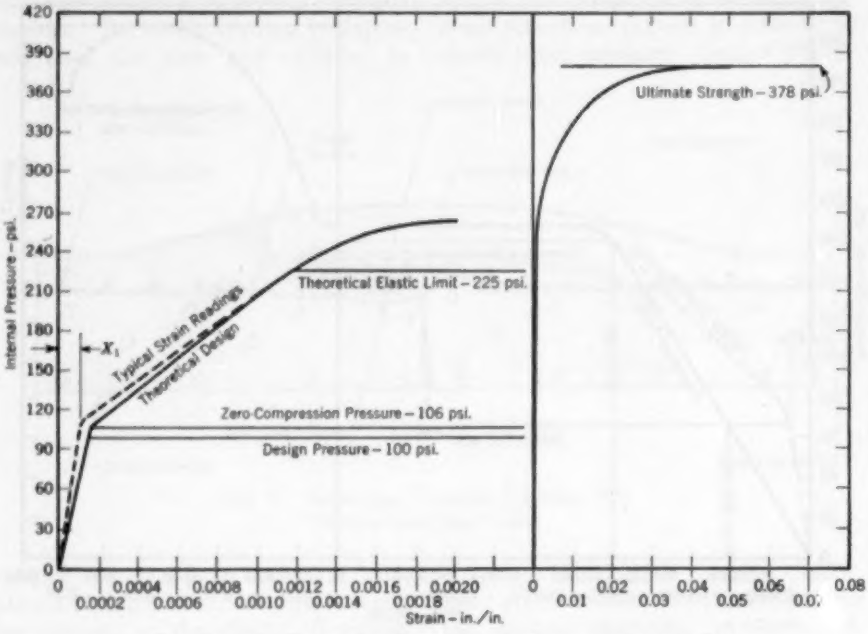


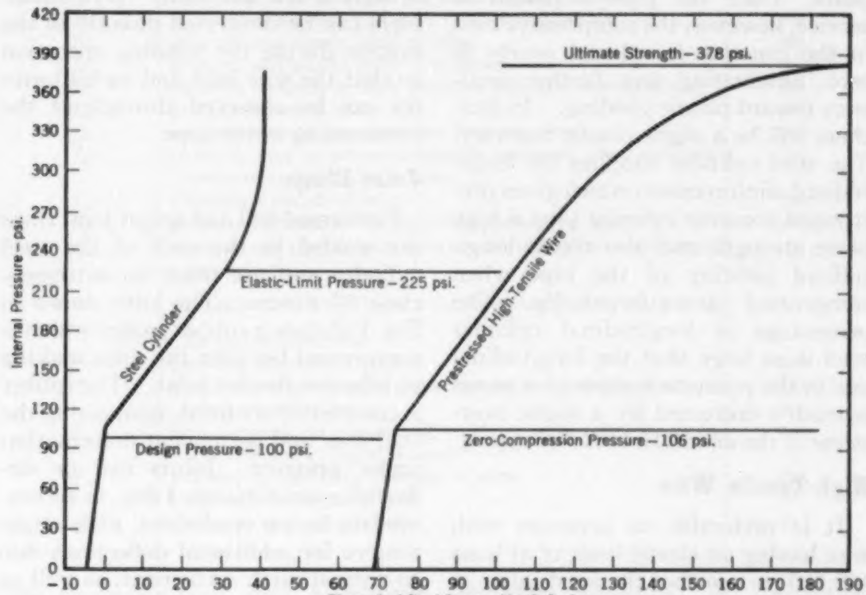
FIG. 3. Pressure-Strain Curve (Prestressed Concrete Pipe)

Concrete Lining

The core of prestressed concrete cylinder pipe consists of the steel cylinder and its concrete lining. The thickness of this core has been chosen

so that, for all normal designs, the initial compression is within the working range of the concrete in compression. A core wall thickness in the ratio of $\frac{1}{16}$ of the inside diameter is used to fulfill this requirement.

The concrete lining is placed with high-frequency vibration in vertical molds, or by the centrifugal method. It is essential that elastic concrete be produced and that the compressive loads on the concrete operate within this elastic range. The desirable properties of concrete must be produced prior to the prestressing operation. For this reason, 28-day strengths are of little practical interest since prestressing is conducted seven days after casting. In order to ascertain the properties of concrete at the time of wire winding, test cylinders are cast in the same manner and cured with the pipe. Cast specimens will attain a strength of 3,500 psi. in 60 hours, and centrifugated specimens will have approximately 50 per cent greater strength. The cast concrete will have a modulus of elasticity in excess of 3,560,000 psi., while the modulus of the centrifugated concrete will be in excess of 4,750,000 psi. The modular values will be $n = 8$ and $n = 6$, respectively, since the steel has a modulus of 28,500,000 psi. Normal initial compressions will vary with design but will usually lie between 600 and 1,800 psi. The initial concrete compression should not exceed 40 per cent of the concrete strength at the time of wrapping.



* 1 kip equals 1,000 lb.

FIG. 4. Pressure-Stress Curve

Steel Cylinder

The steel cylinder fulfills a multiple purpose in the design of prestressed concrete cylinder pipe. It is a watertight membrane and serves as an

element for distributing the concentrated compressive load under the tensioned wires. Concrete subjected to compressive loads will yield plastically and result in loss of prestress and concrete compression. This loss is minimized by elastic, previously cured or reinforced concrete, among other things. For such concrete, a maximum 10–15 per cent loss can be expected to occur in the first five years. Once the pipe is placed in service, however, the compressive load in the concrete is reduced nearly to zero, eliminating any further tendency toward plastic yielding. In fact, there will be a slight plastic recovery. The steel cylinder supplies the longitudinal reinforcement which gives prestressed concrete cylinder pipe a high beam strength and also resists longitudinal yielding of the pipe when compressed circumferentially. The percentage of longitudinal cylinder steel is so large that the longitudinal loss in the concrete compressive stress is readily corrected by a slight overstress in the circumferential wrapping.

High-Tensile Wire

It is preferable to prestress with wire having an elastic limit of at least 100,000 psi., so that the fixed losses in prestressing will be a small percentage of the applied prestress. For practical reasons, the minimum wire size used in prestressed concrete cylinder pipe has been established as $\frac{1}{8}$ in. A maximum spacing of wires has been set at 1 in., center to center, for 16-gage cylinders. The limitation of spacing is due to the effectiveness of the steel membrane cylinder in bridging adjacent supporting wires without appreciable deflections caused by water pressures above the zero-compression pressure.

Tubular mechanical fasteners have been developed for tying adjacent coils of spring wire together and for the indirect welding of the wire at the pipe ends without impairment of the physical properties of the wire.

Machines for winding the wire on the core produce a uniform helix of any desired spacing. The wire is cold worked and is applied under any designed tension load with a fluctuation of only a few per cent. Dynamometers can be connected directly in the system during the winding operation so that the wire load and its uniformity can be observed throughout the prestressing of the pipe.

Joint Rings

Preformed bell and spigot joint rings are welded to the ends of the steel cylinder and are made to extremely close tolerances. The joint shown in Fig. 1 utilizes a rubber gasket which is compressed between the rings making an effective flexible joint. The rubber is completely confined, minimizing the cold flow and permanent deformation under pressure. Joints can be deflected approximately 1 deg. to accommodate laying conditions, with ample reserve for additional deflections due to normal earth settlement, as well as expansion and contraction due to temperature variation. Joint rings can be welded to the cylinder on a bevel for deflections up to approximately 5 deg. Elbows are made for larger deflections.

Mortar Coatings

The rich mortar coating is applied over the cylinder and high-tensile wires to protect the steel from corrosion. An effective method of application makes use of a traveling machine which sprays the premixed mortar on

TABLE 1
Symbols Used in Formulas

Symbol	Meaning	Unit
A_c	area of concrete lining per lineal foot of pipe wall	sq.in.
A_g	gross area of core per lineal foot of pipe wall ($A_g = 12t = A_c + A_y$)	sq.in.
A_s	area of tension wire per lineal foot of pipe wall	sq.in.
A_y	area of steel cylinder per lineal foot of pipe wall (0.718 sq.in. for 16 gage)	sq.in.
D_y	diameter of cylinder (od.)	in.
E_c	modulus of elasticity of concrete (3,560,000 psi. for poured concrete, 4,750,000 psi. for centrifugal concrete)	psi.
E_s	modulus of elasticity of steel (28,500,000 psi.)	psi.
El_s	proportional elastic limit of wire	psi.
El_y	proportional elastic limit of cylinder	psi.
f_{ci}	stress in concrete at zero pressure (initial concrete stress)	psi.
f_{sg}	gross prestress in wire	psi.
f_{si}	stress in wire at zero pressure (net prestress)	psi.
f_{so}	stress in wire at zero-compression pressure, when stress in concrete and cylinder is zero	psi.
f_{su}	ultimate strength of wire	psi.
f_{yb}	stress in cylinder at the bursting pressure of pipe (P_b)	psi.
f_{yi}	stress in cylinder at zero pressure (initial cylinder compressive stress)	psi.
n	modular ratio $= \frac{E_s}{E_c}$ ($n = 6$ for centrifugal concrete, $n = 8$ for poured concrete)	
P_b	bursting pressure of pipe	psi.
P_l	elastic-limit pressure of pipe (pressure at elastic limit of wire and cylinder)	psi.
P_o	zero-compression pressure of pipe (pressure when stress in concrete and cylinder is zero)	psi.
P_w	working or operating pressure of pipe	psi.
t	total wall thickness of core	in.
t_y	thickness of steel cylinder	in.

the revolving pipe with high-speed brushes at a velocity of 6,000 fpm. The resultant coating is dense and uniform in texture and appearance.

Derivation of Design Formulas

The derivation of the standard design formula is based on a hydrostatic analysis. The inherent backload strength of prestressed concrete cylinder pipe is such that average trench conditions are allowed for in this conservative design. Designs for special conditions of crushing and beam loads can be developed by the methods presented in subsequent paragraphs.

The general formula for uniform internal pressure in a pipe is:

$$f_s = \frac{DP}{2T}$$

in which f_s is the tensile stress, D is the pipe diameter, P is the pressure and T is the wall thickness. (The symbols used in the formulas derived below are explained in Table 1.) Since the pipe is to be designed so that the wire and cylinder will reach their respective elastic limits simultaneously,

at the elastic-limit pressure, the following equation can be established (the tensile strength of the concrete is neglected and the water pressure is assumed to act on the cylinder):

$$P_l = \frac{El_s A_s + El_y A_y}{6D_y} \quad \text{...Eq. 1}$$

Solving for A_s:

$$A_s = \frac{6P_l D_y - El_y A_y}{El_s} \quad \text{...Eq. 2}$$

A limitation on the operating pressure is:

$$P_w \leqq \frac{P_l}{2.25} \quad \text{.......Eq. 3}$$

The cylinder and wire remain in intimate contact. Under varying pressures their strain variation is identical, and, since they possess the same modulus of elasticity, their stress variation is identical. At the pressure where the stress in the cylinder is zero, the stress in the wire may be calculated since the wire and cylinder reach their respective elastic limits simultaneously. This stress will be equal to the difference between the elastic limits of the two materials:

$$f_{so} = El_s - El_y$$

At the zero-compression pressure, the high-tensile wire is the only element resisting internal pressure. By definition, the stress in the concrete core and steel cylinder is zero, so that:

$$P_o = \frac{f_{so} A_s}{6D_y} \quad \text{.......Eq. 4}$$

A second limitation on the working pressure is that:

$$P_w \leqq P_o \quad \text{........Eq. 5}$$

The bursting pressure of the pipe may be expressed thus:

$$P_b = \frac{A_s f_{su} + A_y f_{yb}}{6D_y} \quad \text{...Eq. 6}$$

In the above equation, f_{yb} does not equal the tensile strength of the cylinder because of its greater ductility. The bursting pressure of the pipe will occur when the high-tensile wire reaches its maximum elongation. At this elongation, the more ductile cylinder is not at its maximum extension and stress.

As the internal pressure is reduced from the point of zero compression, the concrete, cylinder and wire undergo the same reduction in length or strain. When the internal pressure is reduced to zero, the resistance offered by the concrete and cylinder in compression will be exactly equal to the load of the tensioned wires. The change in strain of the various elements will be:

$$\text{Concrete strain} = \frac{f_{ci}}{E_c}$$

$$\text{Wire strain} = \frac{f_{so} - f_{si}}{E_s}$$

$$\text{Cylinder strain} = \frac{f_{yi}}{E_s}$$

Since the strain in the concrete, cylinder and wire is identical:

$$\frac{f_{ci}}{E_c} = \frac{f_{so} - f_{si}}{E_s} = \frac{f_{yi}}{E_s}$$

Therefore:

$$f_{yi} = f_{ci}\frac{E_s}{E_c} = nf_{ci} \quad \text{.....Eq. 7}$$

And:

$$f_{si} = f_{so} - nf_{ci} \quad \text{.......Eq. 8}$$

By equating loads at zero pressure:

$$A_c f_{ci} + A_y f_{yi} = A_s f_{si}$$

And since:

$$f_{yi} = nf_{ci} \quad \text{........Eq. 7}$$

Then:

$$A_c f_{ci} + A_y n f_{ci} = A_s f_{si}$$

Solving for f_{ci}:

$$f_{ci} = \frac{A_s f_{si}}{A_c + A_y n}$$

$$= \frac{A_s f_{si}}{12(t - t_y) + 12 t_y n}$$

$$= \frac{A_s f_{si}}{12t + (n - 1)A_y}$$

$$= \frac{A_s f_{si}}{A_g + (n - 1)A_y} \ldots \text{Eq. 9}$$

Since:

$$f_{si} = f_{so} - n f_{ci} \ldots\ldots \text{Eq. 8}$$

Substituting for f_{si} in Eq. 9:

$$f_{ci} = \frac{A_s(f_{so} - n f_{ci})}{12t + (n - 1)A_y}$$

$$f_{ci}(12t + (n - 1)A_y) + f_{ci} n A_s = A_s f_{so}$$

Therefore:

$$f_{ci} = \frac{A_s f_{so}}{A_g + (n - 1)A_y + nA_s} \ldots \text{Eq. 10}$$

Since:

$$f_{si} = f_{so} - n f_{ci} \ldots\ldots \text{Eq. 8}$$

And:

$$f_{ci} = \frac{A_s f_{so}}{A_g + (n - 1)A_y + nA_s} \ldots \text{Eq. 10}$$

Substituting for f_{ci} in Eq. 8:

$$f_{si} = f_{so} - \frac{nA_s f_{so}}{A_g + (n - 1)A_y + nA_s}$$

Or:

$$f_{si} = \frac{f_{so}}{1 + \dfrac{nA_s}{A_g + (n - 1)A_y}} \ldots \text{Eq. 11}$$

A minimum 20 per cent overstress in the wire is originally applied to overcome losses, as previously mentioned. The gross prestress will be:

$$f_{sg} = 1.15 f_{si} \ldots\ldots \text{Eq. 12}$$

Below the pressure which produces zero compression in the core (P_o), the concrete, wire and cylinder are all within their elastic limits, and a straight-line distribution of stress against strain is produced for any pressure up to P_o. The stresses in concrete, wire and cylinder may therefore be found by direct proportions.

Above the pressure which produces zero compression in the core (P_o), it is assumed that the concrete will carry no tension. For any pressure between P_o and the elastic-limit pressure (P_l), the stress in the wire and cylinder may be found by direct proportions, since the steel follows a straight-line stress-strain relationship.

For easier reference the principal design formulas are listed below:

$$P_w \leqq P_o; \quad P_o = \frac{f_{so} A_s}{6D_y}$$

$$P_w \leqq \frac{P_l}{2.25}; \quad P_l = \frac{El_s A_s + El_y A_y}{6D_y}$$

$$P_b = \frac{A_s f_{su} + A_y f_{yb}}{6D_y}$$

$$A_s = \frac{6P_l D_y - El_y A_y}{El_s}$$

$$f_{ci} = \frac{A_s f_{so}}{A_g + (n - 1)A_y + nA_s}$$

$$= \frac{A_s f_{si}}{A_g + (n - 1)A_y}$$

$$f_{si} = f_{so} - n f_{ci}$$

$$= \frac{f_{so}}{1 + \dfrac{nA_s}{A_g + (n - 1)A_y}}$$

Crushing Strength

The high crushing strength of prestressed concrete cylinder pipe is due to the induced compression in the

concrete core. Depending on the extent of this compression, the effective tensile strength of the concrete for resisting positive bending moments (producing tension on the inside of the pipe wall) has been greatly increased. For example, assuming a conservative tensile strength in the concrete of 300 psi. and an initial compression of 1,500 psi., the effective tensile strength of the concrete has been increased from 300 to 1,500 + 300 psi., or six times, before initial cracking will occur.

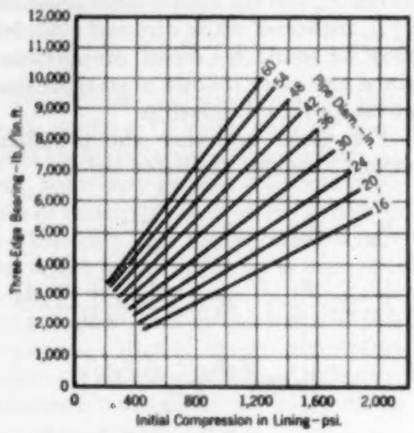


FIG. 5. Three-Edge Bearing Load—First Crack

Where negative moments occur, the location of the steel reinforcement is so placed that it will have an optimum effect.

Crushing strengths for design purposes are plotted in Fig. 5–7. These conservative data, based on tests conducted during the past ten years, record first crack, 0.01-in. crack and ultimate strength. The first crack, which has been established as 0.001 in. wide and 1 ft. long, is measured with a microscope having a grid calibrated in 0.001-in. divisions. Other definitions and methods of loading are consistent with A.S.T.M. requirements. The three-edge bearing load in pounds per lineal foot of pipe is plotted against the initial compression for pipe 16–60 in. in diameter. Specific tests may exceed the minimum loads indicated by as much as 40 per cent, which is accounted for by the variation in concrete materials used in pipe manufactured throughout the United States and South America.

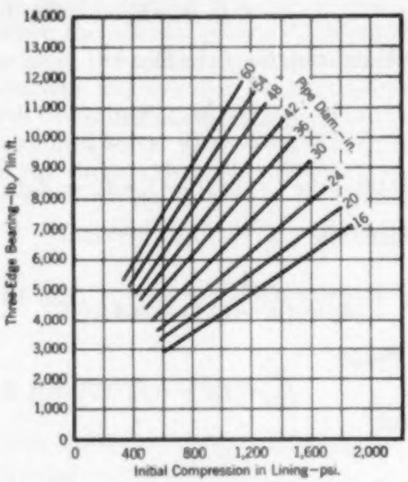


FIG. 6. Three-Edge Bearing Load—0.01-in. Crack

Beam Strengths

Beam tests have been made on 16–36-in. pipe. All specimens were tested by a concentrated, centrally applied load on a 14-ft. span measured from center to center of the end support blocks. The end supports and center load bearing blocks were shaped to the exterior of the pipe for an arc of 90 deg. Figure 8 gives conservative design data for first (0.001-in.) crack, 0.01-in. crack and ultimate load. The concentrated, centrally applied load

in pounds is plotted against pipe diameter. The cylinder in all pipe tested was the minimum 16 gage; special designs using heavier gages will have appreciably greater beam strengths.

The Underwriters' Laboratories, Inc., of the National Board of Fire Underwriters, tested a 20-in. prestressed concrete cylinder pipe to failure as a beam with a concentrated and centrally applied load of 56,970 lb. The pipe was supported on 14-ft. centers and had a centerline deflection of 2 in. at the time of failure. The destroyed pipe was then tested hydro-

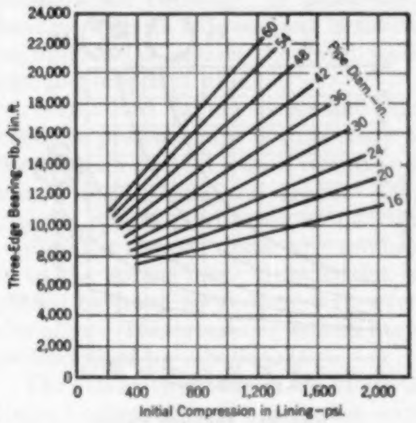


FIG. 7. Three-Edge Bearing Load—Ultimate Strength

statically to 750 psi. without failure or leakage. This pressure was above its original theoretical bursting pressure.

Internal and External Loads

Tests have been made on pipe subjected to combined internal and external loads, conditions which are of primary interest to the designer. A series of these tests has been started to determine the behavior of this type of "semirigid" pipe when subjected to combined loading. Internal bulkheads are used which are designed to give no support to the pipe. Lucite windows are located in the bulkhead for the observation of initial and subsequent cracks in the concrete lining. The bulkheaded pipe is subjected to three-edge bearing loads, and, by varying internal pressure and external load on a series of identical pipes, combination load curves can be derived for first crack, 0.01-in. crack and ultimate strength. The consistent test data

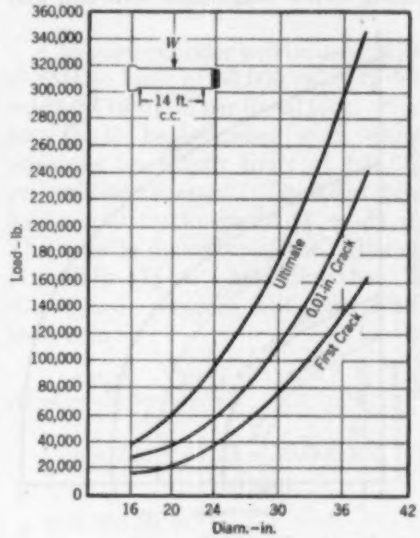


FIG. 8. Beam Strength

indicate a definite trend which, if applied with judgment, becomes a valuable tool for the engineer.

Tests indicate that parabolic curves of a high power may eventually be justified, when sufficient empirical data have been accumulated. Until that time, however, a cubic parabola can be used with conservatism, as illustrated in Fig. 9. The equation of

these curves is:

$$w_1 = \frac{W}{\sqrt[3]{P}} \sqrt[3]{P - p}$$

in which w_1 is the three-edge bearing load (in pounds per lineal foot of pipe) at internal pressure, p (pounds per square inch); W is the three-edge bearing load (in pounds per lineal foot of pipe) at $p = 0$; and P is internal pressure (in pounds per square inch) at $w_1 = 0$.

The design curves can be established if the maximum ordinate and

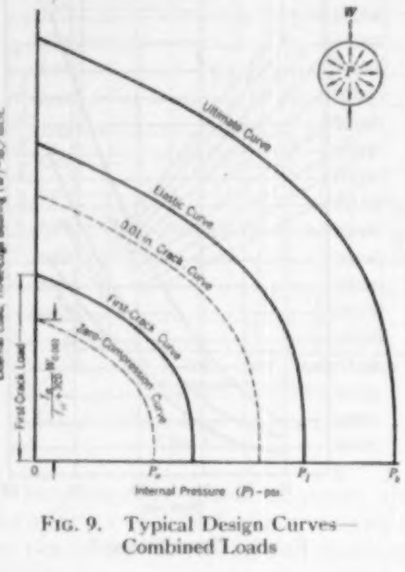


FIG. 9. Typical Design Curves—Combined Loads

abscissa of each are known. For the first (0.001-in.) crack curve, the maximum abscissa or internal pressure will be assumed as the pressure which produces a theoretical tensile stress of 300 psi. in the concrete lining. This pressure will be a function of the zero-compression pressure:

$$P_{0.001} = P_o \frac{f_{ci} + 300}{f_{ci}}$$

The maximum ordinate or external load can be obtained from recorded design data (Fig. 5).

The ultimate curve can be established between the calculated bursting pressure and the ultimate design crushing strength obtained from Fig. 7. The 0.01-in. crack curve will be proportionally spaced between the 0.001-in. crack and the ultimate curves, the maximum ordinate being obtained from Fig. 6. The elastic-limit curve is proportionally spaced and originates from the theoretical elastic-limit pres-

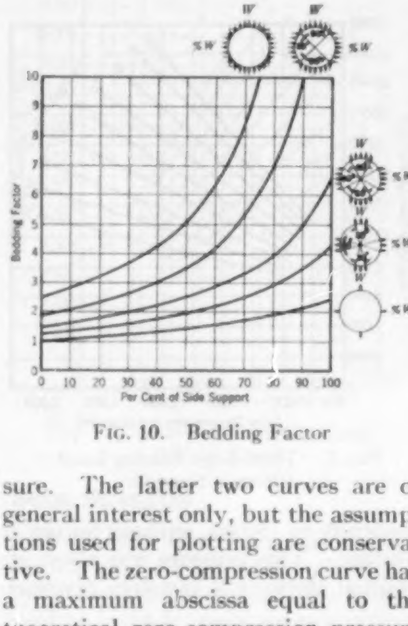


FIG. 10. Bedding Factor

sure. The latter two curves are of general interest only, but the assumptions used for plotting are conservative. The zero-compression curve has a maximum abscissa equal to the theoretical zero-compression pressure and a maximum ordinate equal to the external load (W_o) which has relieved the induced compression in the concrete core. W_o can be expressed as a function of the first-crack load:

$$W_o = W_{0.001} \frac{f_{ci}}{f_{ci} + 300}$$

If, for example, a combination of loads falls below the 0.001-in. crack curve or results in less than 300-psi. tension in the concrete core, the design will be for a condition of no incipient cracking in the concrete. Figure 9 is based on internal pressure and three-edge bearing loads. For trench conditions, the three-edge bearing loads should be converted into trench loads. Figure 10 presents the theoretical resistance of a ring subjected to various distributions of external load and with varying side support. The strength of the ring is expressed as a bedding factor when two-edge, or three-edge bearing with no side support is taken as unity. (W equals the total load per unit length of pipe.)

A standard laying procedure for concrete pressure pipe is to prepare a level trench bottom free of stones and lumps and to line and grade. The pipe is placed, and selected backfill material is tamped under and around it to the spring line. Selected fill is placed by hand to at least 1 ft. over the pipe. The remaining trench may be backfilled by a bulldozer.

The extensive studies made at Iowa State College (2) will supply the necessary data for computing the actual total trench load on the pipe. Converting the three-edge bearing loads to feet of cover, the graphic method of analyzing the pipe subjected to combined internal and external loads can be adopted for design purposes.

Example of Design

For illustrative purposes, the design factors for a 42-in. prestressed concrete cylinder pipe, to operate at 100 psi., will be calculated. The resultant crushing, beam and combination load characteristics will be analyzed. The pressure-strain and pressure-stress curves for this design have previously been shown in Fig. 3 and 4. (The symbols used in the equations below are explained in Table 1.)

The area of high-tensile steel will be fixed by the zero-compression or the elastic-limit pressure, whichever produces the greater area:

$$P_o \geqq 100 \text{ psi.}$$

Or:

$$P_l \geqq 2.25 \times 100 \text{ psi.} = 225 \text{ psi.}$$

A 16-gage cylinder will be used, with an elastic limit of 30,000 psi.; A_y will equal 0.718 sq.in. per lineal foot. Six-gage M.B. basic spring wire, which possesses an elastic limit of 103,680 psi., will be assumed. The 42-in. pipe has a wall thickness of $\frac{21}{8}$ or $2\frac{5}{8}$ in. The outside diameter of the cylinder, D_y, will be $47\frac{1}{4}$ in. Assuming that the elastic-limit pressure will control the steel area:

$$A_s = \frac{6P_lD_y - El_yA_y}{El_s}$$

$$= \frac{6 \times 225 \times 47.25 - 30{,}000 \times 0.718}{103{,}680}$$

$$= 0.408 \text{ sq.in.}$$

This area of high-tensile wire will produce a zero-compression pressure:

$$P_o = \frac{f_{so}A_s}{6D_y}$$

$$= \frac{(103{,}680 - 30{,}000)0.408}{6 \times 47.25}$$

$$= 106 \text{ psi.}$$

This pressure is satisfactory since it is greater than the 100 psi. stipulated.

For calculating the bursting pressure, it can be shown that the stress in the ductile cylinder will be 40,000 psi. at an elongation corresponding to the maximum elongation of the less ductile M.B. wire. Then $f_{yb} = 40{,}000$ psi. The wire possesses a tensile strength of 192,000 psi. The bursting pressure will be:

$$P_b = \frac{A_s f_{su} + A_y f_{yb}}{6D_y}$$
$$= \frac{0.408 \times 192{,}000 + 0.718 \times 40{,}000}{6 \times 47.25}$$
$$= 378 \text{ psi.}$$

The initial compression in the centrifugal concrete lining will be:

$$f_{ci} = \frac{A_s f_{so}}{A_g + (n-1)A_y + nA_s}$$
$$= \frac{0.408(103{,}680 - 30{,}000)}{12 \times 2.625 + 5 \times 0.718 + 6 \times 0.408}$$
$$= 801 \text{ psi.}$$

The net prestress in the high-tensile wire will be:

$$f_{si} = f_{so} - nf_{ci}$$
$$= (103{,}680 - 30{,}000) - (6 \times 801)$$
$$= 68{,}874 \text{ psi.}$$

The gross prestress will be 20 per cent greater than the net, or 82,650 psi. The dynamometer generally records the tension on two wires. Since a 6-gage wire has a cross-sectional area of 0.02895 sq.in., the dynamometer reading is $2 \times 0.02895 \times 82{,}650 = 4{,}785$ lb.

The above calculations give the basic design data, but the conditions of stress can be summarized. The initial net compression in the cylinder will be $nf_{ci} = 6 \times 801 = 4{,}806$ psi.; at the zero-compression pressure, 106 psi., the stress in the cylinder is zero; at the elastic-limit pressure, 225 psi., the stress is 30,000 psi.; and at the bursting pressure, 378 psi., the stress is 40,000 psi. The initial net prestress in the high-tensile wire is 68,874 psi.; at the zero-compression pressure, the stress in the wire is $f_{so} = El_s - El_y = 103{,}680 - 30{,}000 = 73{,}680$ psi.; at the elastic-limit pressure, the stress is 103,680 psi.; and at the bursting pressure, the stress is 192,000 psi. The initial net compression in the concrete is 801 psi., and the stress in the concrete is zero at the zero-compression

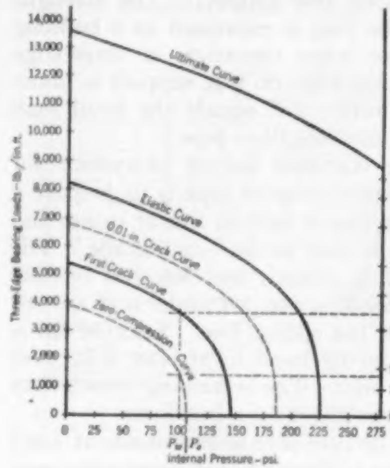


FIG. 11. Combination-Load Analysis

pressure. By direct proportion, the net compressive stress, F, in the concrete at the operating pressure, 100 psi., will be:

$$F = f_{ci}\frac{P_o - P_w}{P_o}$$
$$= 801 \times \frac{106 - 100}{106} = 45 \text{ psi.}$$

The stress in the cylinder and wire at the operating pressure can similarly be obtained, if required.

From Fig. 5–7, the design three-edge bearing loads for this pipe can

be obtained, since the initial compression is established at 801 psi. The 42-in. pipe, when tested according to A.S.T.M. standards, will show incipient cracking (0.001-in. crack) at a load of 5,400 lb. per lineal foot. The 0.01-in. crack will occur above a load of 6,750 lb. per lineal foot and the ultimate strength will be in excess of 13,600 lb. per lineal foot. These loads correspond approximately with those specified for A.S.T.M. extra-strength reinforced concrete culvert pipe (3).

From Fig. 8, the beam strengths of the 42-in. pipe, with 16-gage cylinder, can be obtained. By extrapolation, the concentrated, centrally applied loads are 210,000 lb. for first crack, 320,000 for 0.01-in. crack and 440,000 for ultimate strength. If the pipe is to be supported on pile bents, for example, and one bent is used per 16-ft. pipe length, the pipe will support the following uniformly distributed loads: 184 tons for first crack, 280 for 0.01-in. crack and 385 for ultimate strength.

A combination-load analysis of the 42-in., 100-psi. prestressed concrete cylinder pipe is graphically illustrated in Fig. 11. Cubic parabolic curves of the form:

$$w_1 = \frac{W}{\sqrt[3]{P}} \sqrt[3]{P - p}$$

are plotted on coordinates of three-edge bearing and internal pressure. The ultimate curve is established, since $W = 13{,}600$ lb. per lineal foot (from Fig. 7) and $P = 378$ psi. (from the hydrostatic analysis). The 0.001-in. crack curve is established from $W =$ 5,400 lb. per lineal foot (from Fig. 5) and:

$$P = P_o \frac{f_{ci} + 300}{f_{ci}}$$

$$= 106 \times \frac{801 + 300}{801} = 146 \text{ psi.}$$

The zero-compression curve is established from W, which is a function of the "first-crack load" (Fig. 5):

$$W = \frac{f_{ci}}{f_{ci} + 300} W_{0.001}$$

$$= \frac{801}{801 + 300} \times 5{,}400$$

$$= 3{,}929 \text{ lb./lin.ft.}$$

and from the zero-compression pressure of the hydrostatic design, $P = 106$ psi. The elastic curve has been spaced proportionally between the previous curves and has a maximum abscissa of $P = 225$ psi. (from the hydrostatic analysis). The 0.01-in. crack curve is similarly proportioned and has a maximum ordinate of $W = 6{,}750$ lb. per lineal foot (from Fig. 6).

For analysis of trench loads, the weight of fill will be assumed as 120 lb. per cubic foot; the width of the trench at the top of the pipe will be taken as 6½ ft. and the coefficients of sliding and internal friction will be assumed equal to 0.130.

From the extensive work of Anson Marston (2) and his associates at Iowa State College, the weight of fill on the pipe can be calculated, for any depth of cover, from the equation:

$$W_c = C_d w B_d^2$$

in which W_c is the vertical load on a closed conduit due to fill material, in pounds per foot of length; w is the unit weight of fill materials, in pounds per cubic foot; B_d is the horizontal breadth of trench at top of conduit, in feet; and C_d is a load calculation coefficient for conduits completely buried in ditches. (This is an abstract number —*see* Marston (2).)

Referring to Fig. 10, and using the standard laying procedure previously mentioned, a conservative bedding factor of 2.0 has been selected for the

"semirigid" prestressed pipe. The weight of fill from 10-ft. cover, for example, results in a load on the pipe of 6,080 lb. per lineal foot, according to the Iowa studies (2). With a bedding factor of 2, this represents a three-edge bearing load of 3,040 lb. per lineal foot. The right-hand ordinate of Fig. 11 plots the corresponding depths of cover for the three-edge bearing loads of the combination curves.

From Fig. 11, the standard designed 42-in. prestressed pipe under 100-psi. internal pressure will support a cover of 12 ft. before first crack or 300-psi. tension in the concrete occurs. It will be observed that some compression remains in the concrete core under a design of 100-psi. pressure and 4 ft. of cover. Assuming a ditch condition with 15 ft. of cover, this Class 100 pipe would be recommended for an operating pressure of approximately 75 psi. Tests to date indicate that this combination analysis is very conservative.

Additional Information

For additional information on prestressed concrete cylinder pipe, reference may be made to A.W.W.A. specifications (1) and to other published articles (4, 5). Several papers (6–9) may be referred to on the subject of tests of prestressed concrete pipe (*see* p. 1065, *this issue*).

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Tests of Prestressed Concrete Cylinder Pipe

By F. Eugene Seaman

A paper presented on May 24, 1950, at the Annual Conference, Philadelphia, by F. Eugene Seaman, Research Engr., Lock Joint Pipe Co., East Orange, N.J.

PRESTRESSED concrete cylinder pipe, fully utilizing the stress limits possible in the component parts of the structure, must employ special stressing steel if a high percentage of the original prestress is to remain for elastic work deflections to resist applied forces. The concrete must possess, as nearly as possible, a straight-line stress-strain relationship and a definite endpoint in regard to plastic flow under the sustained precompressive stress. It is evident that the quality of the component materials must be carefully controlled if the pipe is to function in service as designed.

All materials used in the manufacture of prestressed pipe are tested to insure that the completed pipe will give satisfactory performance in service. Tension specimens cut from the cylinder sheet are tested to determine the yield stress. The sheets are checked daily for thickness requirements, and weld seams are examined both for mechanical strength and watertightness. Prestressing wire is periodically tested to determine the physical properties for each particular heat number of steel represented, if not given in mill test reports. Concrete aggregates are tested for compliance with A.S.T.M. standards, and sieve analyses are frequently performed in order to maintain accurate gradation control. Concrete test cylinders are made daily and tested for compressive strength.

The behavior of prestressed concrete pipe can be accurately predicted from the design methods. All of the many destructive hydrostatic tests made have shown results indicated in the original design. These test results, completed over a ten-year period, demonstrate that, with proper production control methods, the performance of prestressed concrete pipe is thoroughly predictable under the various test pressures and service conditions.

Hundreds of crushing tests have been made throughout the last ten years. These test strengths usually are not design considerations in the average prestressed cylinder pipeline, as the minimum hydrostatic design possesses inherent bearing strengths in excess of the A.S.T.M. requirements for culvert pipe. A number of beam load tests have also been performed. The beam strength, because of the steel cylinder, is so great as to require no consideration other than a knowledge of permissible limits for exceptional installation requirements. Routine absorption and specific gravity tests are made on the protective sand-cement-mortar coating, as well as daily mortar mix analyses.

Material Tests

The success of the prestressing concept requires that steel of comparatively high yield strength be used if the concrete (after shrinkage, plastic

flow and so on) is to remain in compressive strain, in order that elastic deformation (by decompression of the concrete) of the pipe may be possible. It has been shown that, because of the relaxation of steel and plastic flow of concrete for given stress magnitudes (and, finally, concrete shrinkage), ordinary reinforcing steel retains very little of the original stress after a few months' time. Since this inherent loss of stress is of a fixed magnitude, it becomes necessary to use a reinforcing steel with a very high elastic limit so that an efficient percentage of the original prestress will remain after the predetermined shrinkage has occurred. Steel possessing these high yield stress qualities is necessarily limited to sizes normally designated as wire.

Stressing Wire

For prestressed concrete cylinder pipe, wire of ¼-in. diameter or No. 6 gage (0.192-in. diameter) has been satisfactory in meeting the requirements of most pipe installations. Unusual installations may require the use of a larger size. Both of the presently used sizes are obtainable in the A.S.T.M. hard-drawn and the oil-tempered steel spring wire classifications with proportional-limit stresses in excess of 100,000 and 160,000 psi., respectively (1, 2). Figure 1a shows the stress-strain diagrams for these wire types and sizes, while the modulus of elasticity and variously defined yield stresses are given in Table 1. Each curve represents the average of five specimens. High yield strength wire, as contrasted with ordinary reinforcing steel, has no definite yield point and, consequently, must be defined by the proportional limit (straight-line function), offset limit and Johnson's elastic limit. The oil-tempered wire, which undergoes the tempering treatment while in a straight position, is wound in coils for shipment, the coils being

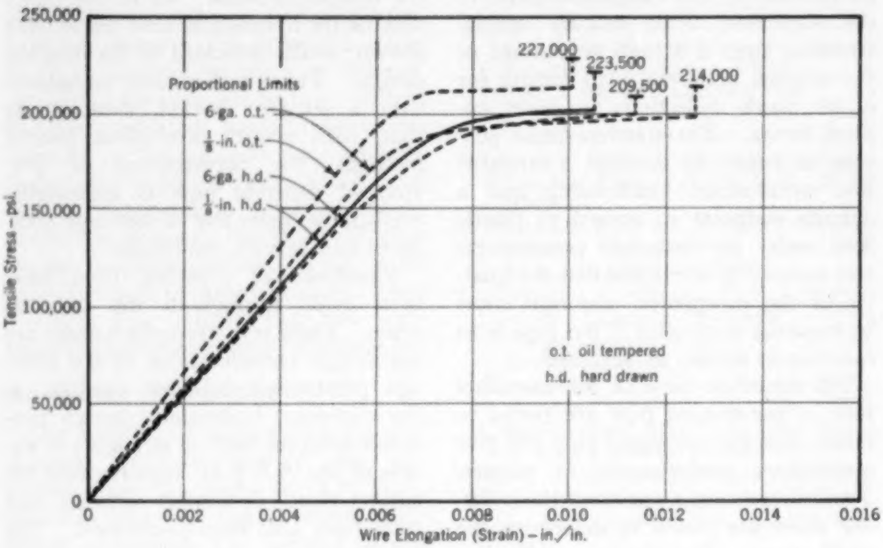

Fig. 1a. Prestressing Wire Stress-Strain Curves

of sufficient diameter to prevent permanent deformation in the circular form. The wire will assume a straight position when released from the roll.

The hard-drawn steel spring grade of wire is permanently deformed in the roll and assumes a coiled or helical position when released. For that reason, straight-line stress-strain curves for this wire are not possible without proof loading above the stress at which the strain function is desired. (In Fig. 1a, the $\frac{1}{8}$-in. hard-drawn wire is proof loaded to 195,000 psi. and the 6-gage, to 180,000 psi.) This proportional stress-strain relationship can be demonstrated by preloading to a definite stress and cyclic reloading to a lesser value so that the permanent set can be observed, if present. Figure 1b represents the load-strain curve as indicated by an electronic autographic recorder with a gage length of 2 in. The first curve represents the original proof load, and the additional curve represents repeated loadings to the proportional elastic-limit stress. This proof loading, in effect, is accomplished in the prestressing machine as the wire passes over the stress-inducing wheel. All deformations which are accompanied by any set would require the prestress loss factor to be increased. Therefore, the proportional limit is the maximum to which the wire stress can be designed if the regular shrinkage factors are to be used.

High tensile strength wire has not been welded to date without a loss in its original strength. For this reason, special tubular steel fasteners which are mechanically deformed in place on the wire are used to join the successive coils of wire and to enable the prestressing wire to be dead-ended by welding the connectors to each joint ring of the pipe. In the latter process, the sleeve is crimped on the wire and subsequently welded. These connectors develop 98 to 100 per cent of the tested ultimate strength of the wire, which usually is at least 10 per cent in excess of the A.S.T.M.-specified minimum ultimate strength of the wire as used in the design.

The wire stress is accurately controlled during the prestressing operation by a dead weight and lever arm system, which, once adjusted for a given wire stress, accurately continues to apply this force with only minor variations at full speed. Remote-indicating or direct-reading dynamometers can be connected in the system so that the actual stress can be observed

TABLE 1

Prestressing Wire Stress-Strain Data

(See Fig. 1a)

Wire		Proportional Limit		Johnson's Elastic Limit		0.2% Offset Limit		Modulus of Elasticity by Proportional Limit
Size	Type	Stress *psi.*	Elongation *in./in.*	Stress *psi.*	Elongation *in./in.*	Stress *psi.*	Elongation *in./in.*	*psi* $\times 10^6$
$\frac{1}{8}$ in.	oil tempered	168,000	0.0056	202,500	0.0063	215,000	0.0085	30.0
$\frac{1}{8}$ in.	hard drawn	121,000	0.0043	187,500	0.0069	200,000	0.0092	28.2
6 ga.	oil tempered	170,000	0.0057	175,000	0.0060	195,000	0.0086	29.8
6 ga.	hard drawn	138,000	0.0053	177,000	0.0067	196,000	0.0091	27.2

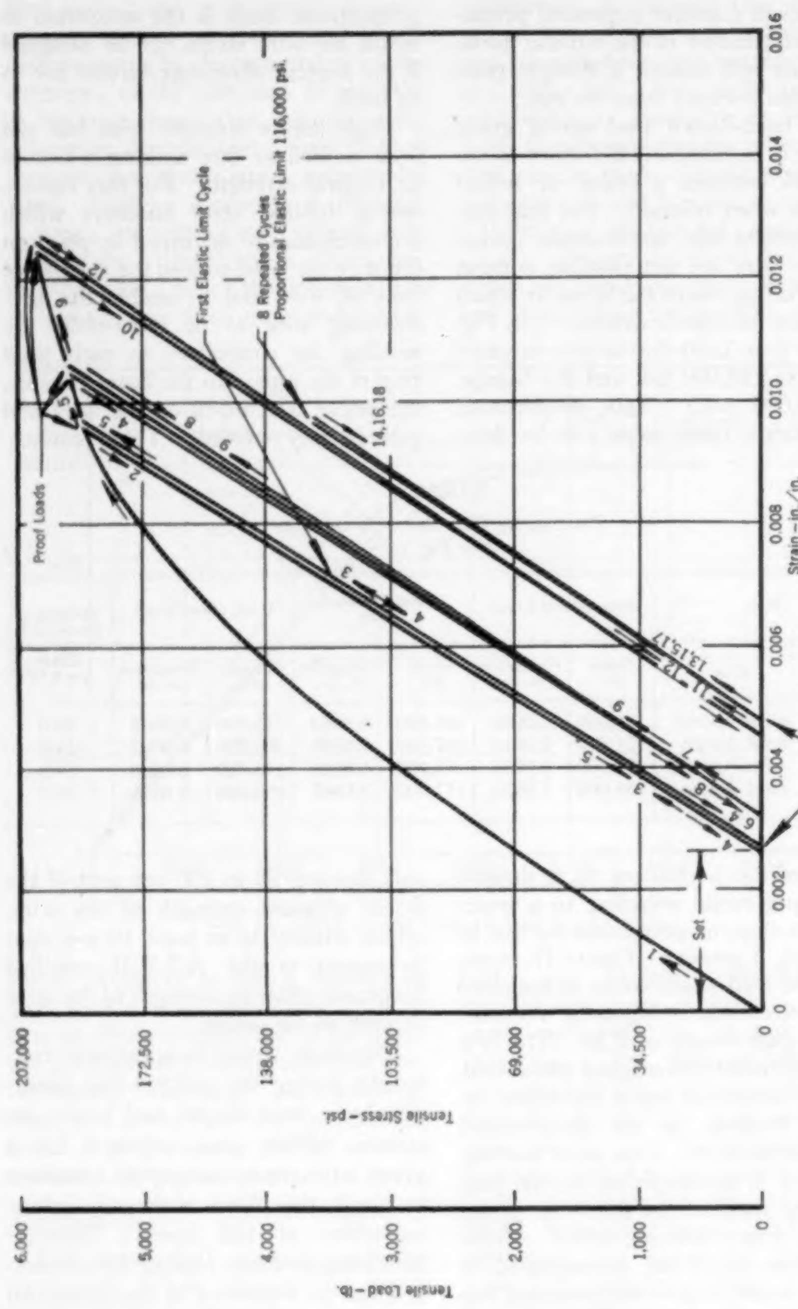

FIG. 1b. Repeated Loadings (No. 6 Hard-drawn Wire)

throughout the entire prestressing operation.

Steel Cylinder

The steel cylinder, in addition to being an absolutely watertight membrane, serves as a structural reinforcement against the longitudinal strains due to the Poisson's ratio effect resulting from the circumferential prestressing and the extreme fiber stresses induced in beam loading conditions. Should the pipe be subjected to pressures above the designed zero compression, the tensile stress of the cylinder opposes the hydrostatic pressure force, along with the prestressing wire. Furthermore, the cylinder provides a secure method of joining the steel joint rings to the pipe structure.

The cylinders are formed of a number of full pipe length steel sheets and the zinc-coated bell and spigot rings are welded on the ends. The completed and automatically welded cylinder is then tested hydrostatically.

Tension tests on reduced-section coupons cut from over the cylinder sheet weld show that the weld junction of the adjacent sheets has a higher ultimate strength than the adjacent metal. The point of ultimate failure is nearly always located midway between the weld and the increased grip section. Tests on plain tension coupons cut from the center of sheets, with weld beads run across prior to cutting the coupon, indicate that the annealing of the parent metal has no appreciable effect on the yield strength and elastic elongation properties of the plain sheet. Hot-rolled 16-gage steel sheets are used for the cylinder, but special alloy steel sheets may be used if greater strength is required, and heavier-gage sheets may be used for exceptional conditions of design.

Figure 2 illustrates a typical stress-

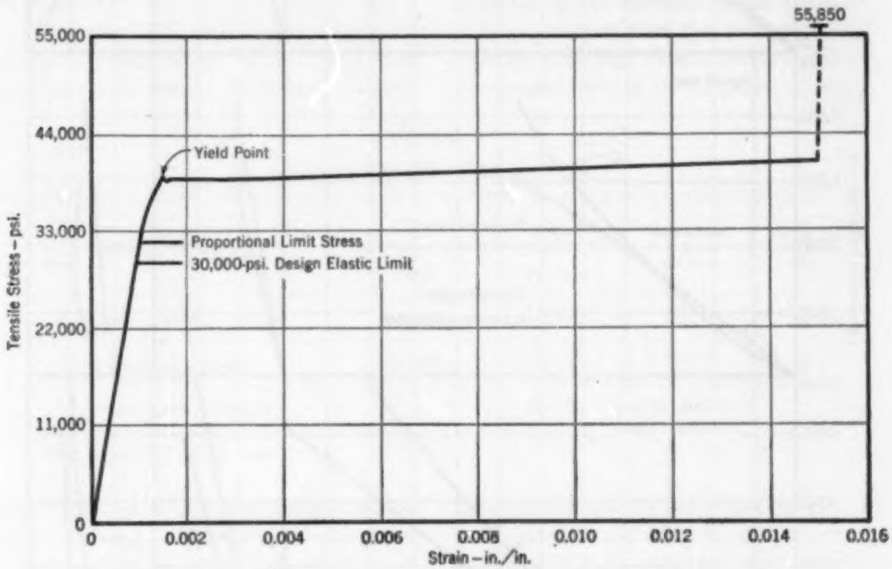

FIG. 2. Cylinder Sheet Stress-Strain Graph

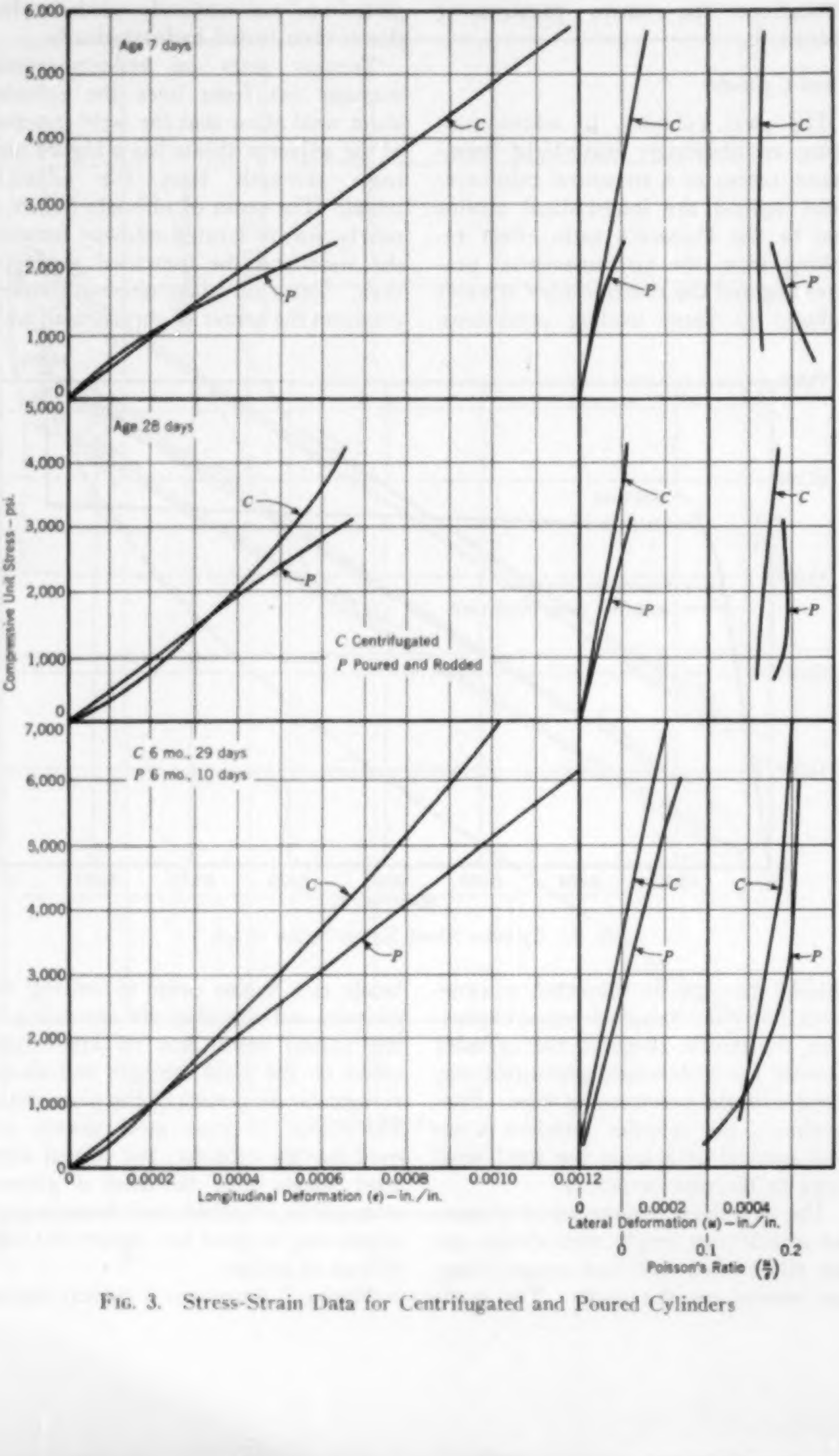

FIG. 3. Stress-Strain Data for Centrifugated and Poured Cylinders

strain relationship (average of three specimens) for hot-rolled 16-gage cylinder sheet steel (2-in. extensometer, 2-in. gage length). The yield point (by halt in dial) is 38,300 psi., the ultimate strength is 55,850 psi. and the modulus of elasticity is 29,800,000 psi. (31,040 ÷ 0.00103). Table 2 is a brief summary of the physical properties of a typical cylinder steel sheet (16 gage).

Joint Rings

The steel bell and spigot rings are made of flat stock, rolled and butt welded and finally stretched circumferentially in a press designed to expand the ring to the specified diameter. Thus, all joint ring butt welds are tested above the yield stress. The joint rings are the only portion of the pipe steel structure not protected by mortar or concrete in the pipe and, therefore, are galvanized or zinc metallized. The exposed portion of the joint is mortared after installation, and, in this way, the entire pipe is protected by mortar or concrete from corrosion.

Concrete Core

The concrete lining, in addition to protecting the inside of the cylinder from corrosion, provides a smooth surface with efficient hydraulic properties and is the structural backbone of the pipe. The concrete core and the steel cylinder are initially compressed circumferentially by the prestressing wire in accordance with the design principles. Then, as the internal hydrostatic pressure is increased, the concrete compressive stress is gradually supplanted by the pressure force, so that, at the operating pressure, a small compressive stress remains in the concrete.

It is evident that the concrete must possess excellent elastic qualities if the composite unit is to continue to function as assumed for the pipe design. It has been demonstrated that concrete of excellent elastic qualities will have a comparatively greater modulus of elasticity and will have less divergence of the secant modulus from the tangent modulus. The stress-strain curve is almost a straight-line function up to stresses exceeding those employed in the design of prestressed concrete cylinder pipe.

In determining the strain data, both poured and centrifugally cast concrete cylinders were used, in 3×6- and 6×12-in. sizes. For the cylinders of 6-in. diameter, a specially designed apparatus was developed which measured

TABLE 2

Test Data for Steel Cylinder Sheet

Stress or Strength	Tensile Strength—*psi.*		Reduction of Area *per cent*	Elongation in 2 in. *per cent*
	Yield	Ultimate		
Circumferential stresses	33,450	56,800	51.2	32.2
Longitudinal stresses	32,900	56,740	57.2	33.2
Machine weld strength		54,700	*	*

* Failure occurred outside weld.

the strain in the central 8-in. gage length, averaged the strain for four elements at 90-deg. displacement, multiplied this average strain and indicated the final strain to $\frac{1}{30,000}$ in. If expressed on a unit basis, this would give an accuracy within $\frac{1}{240,000}$ in. Simultaneously a similar device (but nonaveraging) measured the lateral expansion of a diameter of the cylinder to an accuracy within $\frac{1}{300,000}$ in. The strain data obtained with this equipment have been

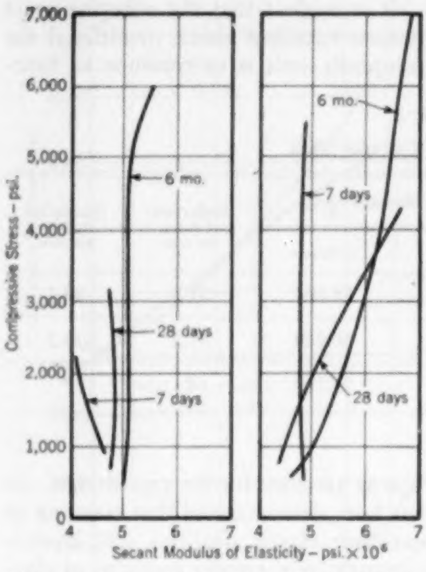

FIG. 4. Secant Modulus and Compressive Stress

plotted on Fig. 3, which shows the 7-day, 28-day and 6-month age stress-strain functions and derived modulus of elasticity of 3 × 6-in. centrifugated cylinders and 6 × 12-in. poured and rodded cylinders. The lateral strain is also indicated for various increasing compressive-stress values, together with the ratio of lateral (u) to longitudinal (e) strain (Poisson's ratio) plotted against compressive stress. This figure illustrates the greater strength and higher modulus of elasticity that is characteristic of concrete placed by the centrifugal method, as compared with the same concrete placed by the usual pouring methods.

These tests were performed on a random selection of cylinders and represent various concrete mixes and curing methods. With variations of this type, it was to be expected that all of the physical properties thus determined would not form a regular sequence in magnitude, but would indicate minimum properties which are of primary consideration. An extensive basic research project, to be completed some time in 1951, has been undertaken, in which, as far as possible, only one of the many variables affecting concrete strength is permitted to change at one time.

In Fig. 4, the secant modulus of elasticity is plotted against compressive stress for standard rodded concrete cylinders, representing poured and vibrated concrete linings, and for centrifugally cast concrete cylinders, representing centrifugated concrete linings. Some irregularities do exist in these derived moduli of elasticity, as would be expected from small variations in curing and concrete mix proportions. These test cylinders were made over a period of two years and demonstrate the conservative physical properties employed in the design of prestressed pipe.

In Fig. 5, the derived modulus of elasticity for various compressive stresses is expressed in terms of the ultimate strength for both the centrifugated and standard rodded cylinders at the age of seven days, when the prestressing operation is usually performed. This chart shows the modulus of elasticity (E)—whether deter-

mined by the tangent or the secant method—for centrifugated concrete to be in excess of 4.75×10^6; and for poured concrete, to be in excess of 3.56×10^6. Using as a modulus of elasticity for steel a value of 28.5×10^6, a modular ratio of $n = 6$ is proper for the centrifugated concrete, and a ratio of $n = 8$ for the poured or vibrated concrete.

compacted aggregates, except where special variations are required in the centrifugal process. The cement factor is maintained at, or in excess of, 1.75 bbl. of cement per cubic yard of freshly mixed concrete.

Mortar Coating

A sand and cement-mortar coating is applied immediately after the con-

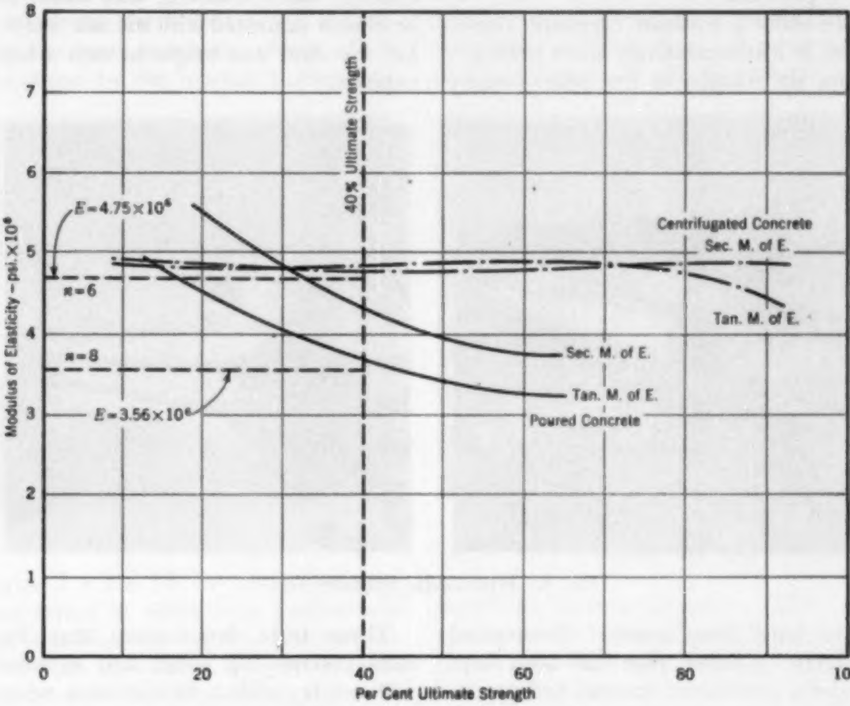

FIG. 5. Elasticity Moduli and Ultimate Strength

Aggregates used in concrete lining are required to meet the A.S.T.M. standards of quality. Although both crushed rock and natural gravels are satisfactory, the latter must be entirely free of soft or low-density particles for use in the centrifugated concrete. The concrete mixes are designed on the basis of maximum density of the fully

crete cylinder pipe is prestressed. The coating protects the prestressing wire and the steel cylinder against corrosion and damage by handling. This mortar is applied by a special process at a zero-slump consistency. By reference to definitions for moisture in soil stabilizations, this moisture would be slightly in excess of the optimum required

for maximum density as determined by the Proctor Mold. The coating is very dense and has an average specific gravity of 2.33. The percentage of moisture absorption is determined by immersing in boiling water for five hours and cooling, then oven-drying the specimens to a constant weight at 230°F. When cooled, the specimen is reweighed. The absorption will average 7.50 per cent.

In order to evaluate corrosion resistance in a comparatively short period—from six months to five years—many tests have been made. Prestressed concrete cylinder pipe has been kept under a continuous internal hydrostatic pressure up to three times the designed operating pressure and alternately immersed in salt water and dried. These tests have been conducted continuously for periods up to six months. In these severe tests, the coating was found to provide complete protection for the prestress wire and the cylinder under all operating conditions. This fact was determined by a microscopic examination of the wire.

Prestressed pipe has been alternately and continuously immersed in rock salt and calcium chloride solutions for five years. Periodically a small area of the coating is chipped out and the prestressing wire and cylinder are carefully examined. No corrosion has ever occurred where the steel has been protected by the mortar coating or concrete lining. In all pipe examined, the coating, when removed, was found to be almost saturated with the salt water, but the steel was bright as new when exposed.

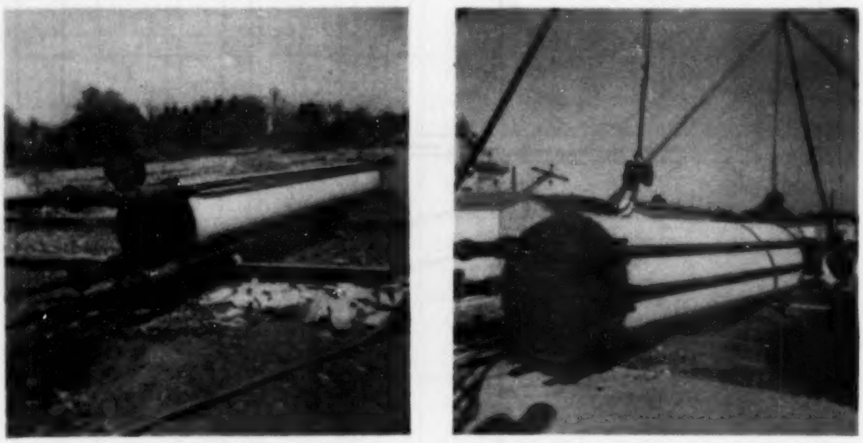

Fig. 6. Hydrostatic Pressure Test

These tests demonstrate that the steel prestressing wires and cylinder will not be subject to corrosion when protected by the alkaline sand-cement mortar. The coating mortar is not impervious, as indicated by the moisture absorption test, but the porosity is so small as to render the moisture completely immobile, and, in this condition, the dissolved or free oxygen necessary for corrosion cannot be replenished. Additional information on the protection afforded by Portland cement bound coatings may be found in a re-

cent article by Pletta, Massie and Robins (3).

Standard briquets, made by filling the mold by the brush method, were compared with briquets made of the same batch but placed in accordance with the standard A.S.T.M. methods. Both methods were found to give identical average tensile strengths of 430 psi. at 28 days for approximately 60 specimens each. Because the briquet method was considered as not indicating the true maximum tensile stress developed by the mortar, beam specimens $2 \times 2 \times 18$ in. were prepared and tested in third-point loading and the modulus of rupture determined. Based on the results of over 20 tests, the average modulus of rupture was 640 psi. Since the third-point loading method in beam strength tests is less subject to eccentricities than the briquet method, the latter ultimate-stress determinations are considered more applicable to the function of the mortar coating on the pipe.

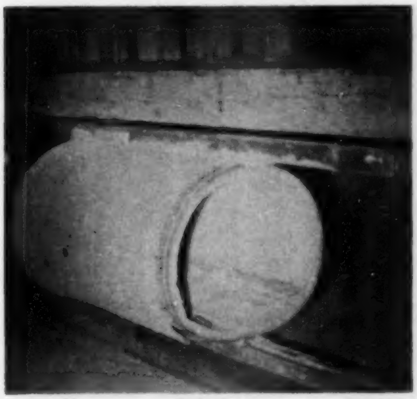

Fig. 7. Bearing Test Specimen Failure

Standard cylinders made of the mortar mix and tamped to a density or degree comparable with the mortar on the pipe show a compressive strength of more than 6,000 psi. in seven days. In view of the results of the tensile- and compressive-strength tests, the brush-placed mortar coating is a tough cement product capable of providing the required protection to the prestressing wire and steel cylinder.

Pipe Production Control

In line with the foregoing material considerations, accurate control of manufacturing processes is constantly maintained. Many tests are grouped, and some qualities are ascertained by significant indirect means. After the steel cylinder has been completed with joint rings, the entire unit is tested by hydrostatic pressure to at least 25,000-psi. circumferential tensile stress. Thus, the steel cylinder of every pipe is pressure tested from the steel spigot groove to the steel bell ring, assuring a watertight pipe and a strength-tested weld.

Fig. 8. Spring Line Prior to Ultimate Load

Control of the aggregates is maintained by routine sieve analysis and other tests, as required, depending on the uniformity of the aggregate supply. Standard concrete test cylinders are

made up daily and tested for 7- and 28-day compressive strengths. Laboratory testing facilities are usually available at each plant site, or contracts are made with local testing and inspection firms for these services. Mix variations of any consequence are usually immediately apparent in the behavior of the concrete during the centrifugal

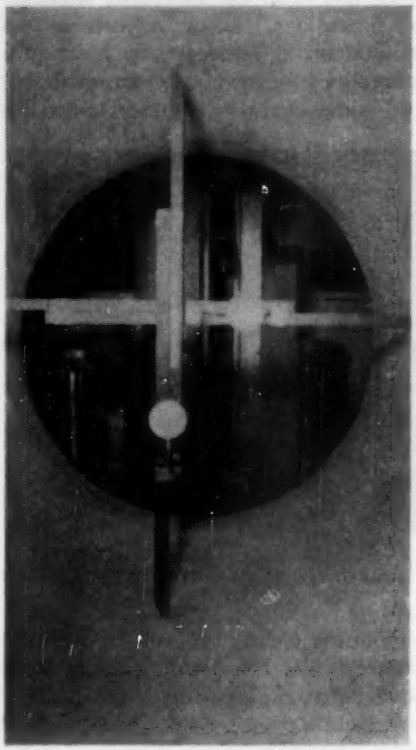

Fig. 9. Arrangement of Gages

operation, or, in poured pipe, are apparent as soon as the form is stripped. The method of wrapping the prestressing wire over the tension-inducing wheel provides a satisfactory proof test of the wire and the sleeve splice.

The prestressed steel cylinder and concrete core are coated with the sand-cement mortar, placed in bins for the final curing and subsequently moved to the storage yard. The entire pipe is inspected and the bell and spigot rings are cleaned of traces of concrete and grease. The completed prestressed concrete cylinder pipe is then ready for shipment to the installation site.

Hydrostatic Pressure Tests

Hydrostatic pressure tests are usually performed on a pipe by bulkheading the bell and spigot ends with a heavily reinforced metal plate to which the mate of the particular joint end has

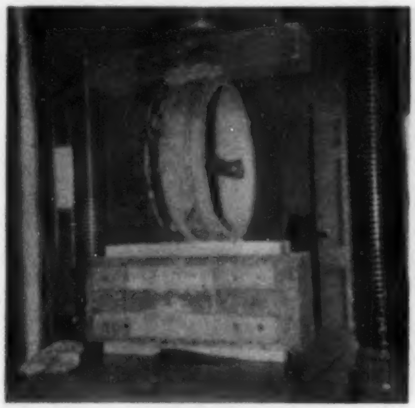

Fig. 10. Spigot Joint Ring Under Bearing Test

been welded. This not only provides a performance test on the pipe, but also a test on two joints identical with those of the installed pipeline. After the bulkheaded pipe has been filled with water, the pressure may be raised immediately to that at which observations are desired. As explained previously, the internal hydrostatic pressure gradually reduces the compressive stress in the concrete core and steel cylinder. The pressure at which these stresses become zero is designated as the zero-compression pressure. The working or operating pressure is such that it

will not exceed the zero-compression pressure, so that a small residual compressive stress will remain under operating conditions.

Figure 6 shows 16-in. (left) and 36-in. (right) prestressed pipe bulkheaded for hydrostatic pressure test.

The operating pressure and the zero-compression pressure, in that order, become the first critical pressures at which observations of the physical behavior are to be made. Above the zero-compression pressure, the concrete core is assumed to carry no tension, and tensile stresses are increased in the steel cylinder, as well as in the prestressing wire, in order to resist hydrostatic forces.

Design considerations usually specify the elastic-limit pressure at 2¼ times the operating pressure. The elastic-limit pressure is the maximum at which the strain in the steel of the pipe is proportional to the hydrostatic pressure producing the strain. The final item to be observed in the complete destructive hydrostatic pressure test is the bursting pressure, at which the ultimate strength of the prestressing wire is developed. For additional hydrostatic tests on prestressed concrete steel cylinder pipe, the test reports of the Underwriters' Laboratories, Inc., and the National Bureau of Standards should be referred to (4, 5).

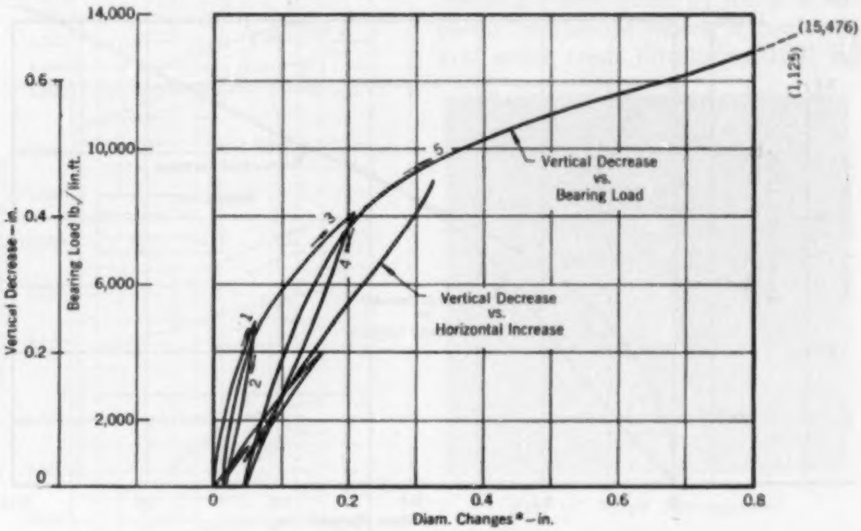

FIG. 11. Three-Edge Bearing Test Performance (Without Joint Rings)

* Horizontal increase, vertical decrease.

Bearing Strength Tests

All pipe specimens were tested in accordance with the three-edge bearing test as given in A.S.T.M. standards for concrete pipe. The two bottom hardwood bearing strips were securely fastened to a 7×9-in. fir timber block. The bearing-edge strips were spaced an inch apart for each foot of the pipe internal diameter. The load was applied at the top of the pipe

through a 3½ × 5½-in. fir wood loading block with the 5½-in. face in contact with the pipe. Both the bottom two bearing-edge strips and the top loading block were capped with a neat cement and plaster of Paris mixture to provide even support throughout the length of the pipe. In the tests where the bell and spigot rings remained on the specimen, these were load supported by contouring the two edge strips and the top loading block before applying the capping mixture.

The load was applied by means of a hydraulic-ram type of testing machine (300,000-lb. capacity) through a heavy continuous steel beam which was pivoted at the center to provide equal bearing load on the pipe specimen. The applied load was recorded directly in pounds. In order to measure the deformation on the pipe specimen, two Ames * dial gages, reading directly to 0.0001 in., were mounted inside the pipe at the center of the test length so as to indicate the decrease in the vertical diameter and the attending increase in the horizontal diameter under the various loadings.

Figure 7 shows a three-edge bearing test specimen after failure by crushing of lining. Figure 8 is a view of the spring line of a pipe just prior to ultimate load. The photograph shows more than 100-deg. arc length of pipe. The dark streaks are due to trapped moisture which wets the surface of the coating as the crack occurs, the actual cracks being almost invisible. An inside view of the pipe, showing crossed diameter gages, may be seen in Fig. 9. A spigot joint ring with concrete lining is shown under bearing test in Fig. 10.

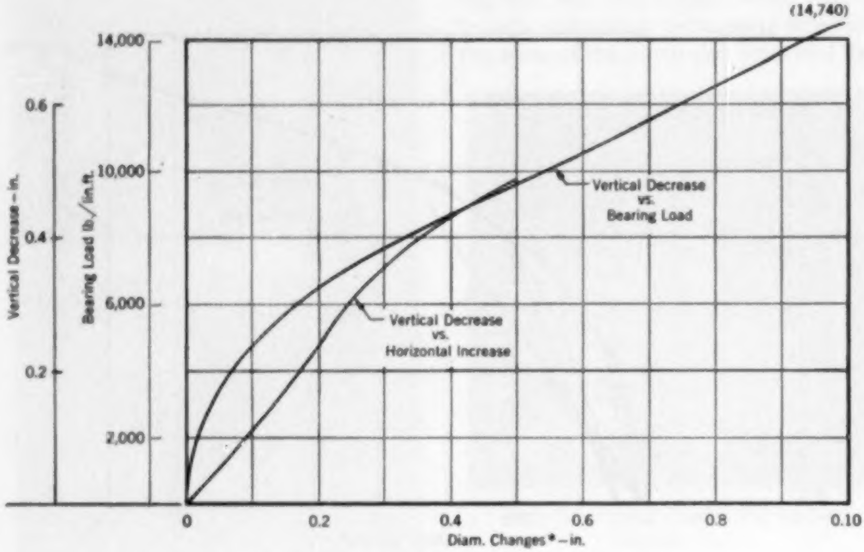

FIG. 12. Three-Edge Bearing Test Performance (With Joint Rings)

* Horizontal increase, vertical decrease.

* B. C. Ames Co., Waltham, Mass.

From the bearing load on the test section of pipe and the effective length, the bearing strength in pounds per lineal foot was determined and then plotted against the vertical diameter change; also, the vertical diameter change was plotted against the horizontal diameter change. Figures 11 and 12 illustrate typical examples of the performance of 30-in., Class 125 prestressed concrete cylinder pipe. Figure 11 also shows the effect of repeated loading and recovery after release of the bearing load.

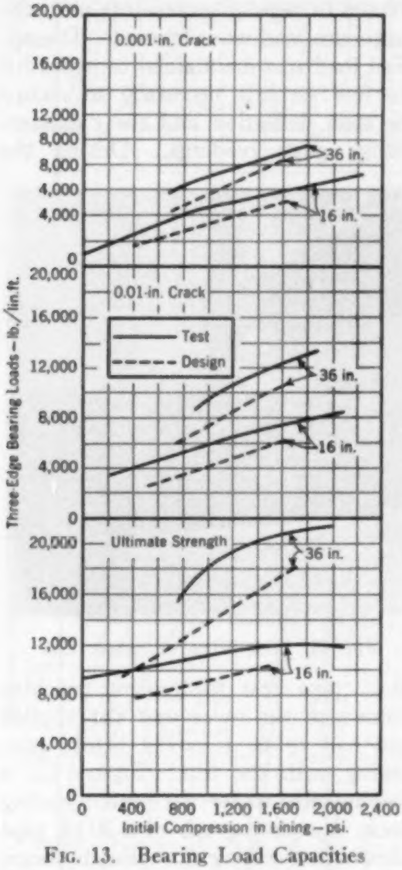

Fig. 13. Bearing Load Capacities

Based upon bearing strength data obtained in the three-edge bearing test, the bearing load capacities for the various lining crack widths were plotted against the concrete initial compressive stress for 16- and 36-in. pipe (Fig. 13). The first crack is defined as a crack in the top or bottom lining 0.001 in. wide and at least 1 ft. long. The 0.01-in. crack is defined as a crack in the top or bottom lining 0.01 in. wide and at least 1 ft. long. The cracks are determined by use of a 40-power microscope having a projected grid which reads directly to 0.001 in.

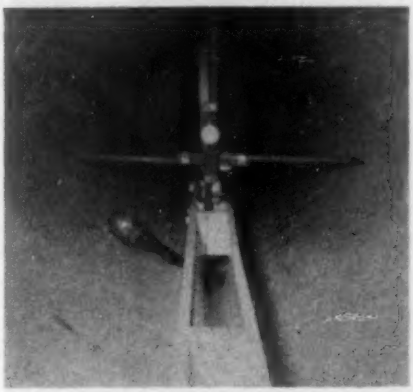

Fig. 14. Gage Arrangement

Also plotted on Fig. 13 are the design curves used in predicting three-edge bearing loads.

Beam Strength Tests

All specimens were tested on a 14.0-ft. span measured from center to center of the end support blocks. The measured center load was applied by means of a hydraulic-ram type of testing machine with a 300,000-lb. capacity. The end support blocks and the center load bearing block were accurately contoured to the external pipe circumference for an arc length of 90 deg.

The width of the support and loading blocks varied with the pipe diameter, as shown in Table 3, which also includes specimen design data. All bearing blocks were capped with a 1:1 mixture of high-early-strength cement and fine sand, and were permitted to set at least 24 hours before testing.

The applied machine load was recorded directly in pounds, while the attending diameter deformation and beam deflection were recorded by means of Ames dial instruments. Decrease in the vertical diameter and increase in the horizontal diameter were indicated by gages reading to 0.0001 in. The beam deflection was measured by means of a rigid wood bridge with pointed steel end supports over the exact center of the two end support blocks (*see* Fig. 14). The gage mounted at the center of the bridge recorded the deflection of the bottom center of the pipe to the nearest 0.001 in. In the 16- and 20-in. specimens, where there was insufficient room for both the crossed diameter gages and the bridge span gage, the deflection of the bottom center of the pipe was measured externally, directly to the floor, with a 0.001-in. gage of 1-in. total travel. It was determined that the floor deflection for loads to 100 kips (100,000 lb.) was negligible.

Fig. 15. Pipe Tested to Ultimate Beam Strength

Fig. 16. Side View of Spigot End

In all pipe beam tests, except for the 36-in. pipe, the applied load was increased in regular increments until the maximum load was obtained. The applied load was maintained constant for the few seconds necessary to record the span deflection and the diameter-change gage readings. During the 36-in. pipe test the testing machine motor cut out twice and the applied load had to be removed before continuing with the test. Figure 15, a closeup view under the center loading block, is a photograph of a 30-in. pipe taken after testing to ultimate beam strength. The stress lines are traces of moisture escaping through surface fractures in the coating. Figure 16 is a side view of the spigot end.

From the data recorded during the test—center load, vertical diameter deformation, horizontal diameter deformation and bottom center deflec-

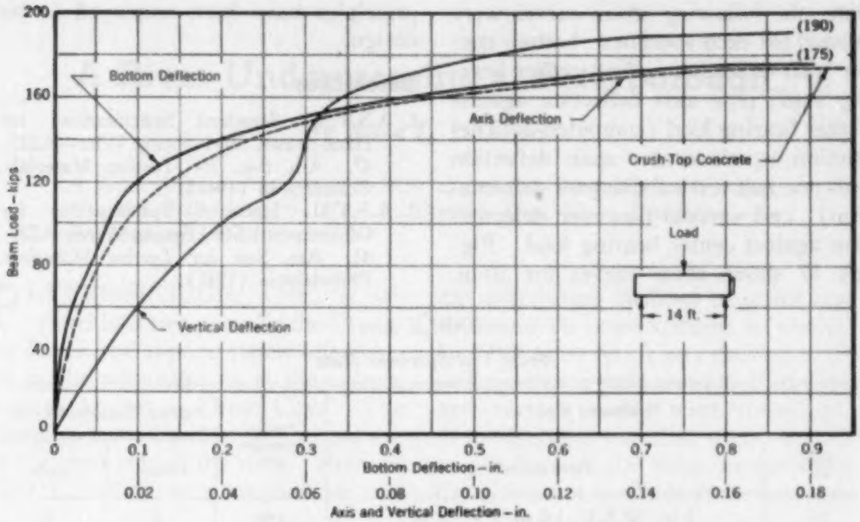

FIG. 17. Beam Test Strain Curves (30-in. Pipe)

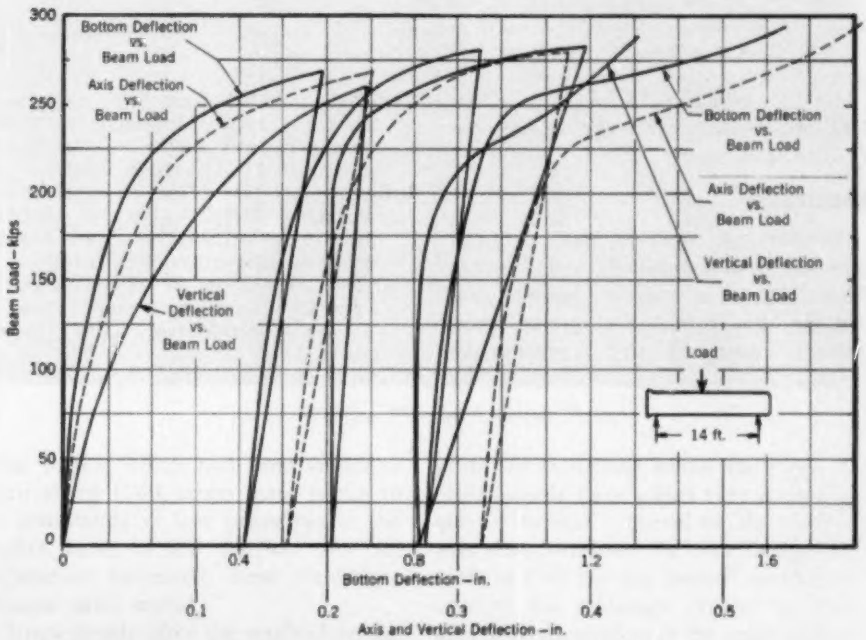

FIG. 18. Beam Test Strain Curves (36-in. Pipe)

tion—the following strain curves were plotted for each specimen: bottom center span deflection against center bearing load; pipe axis deflection against center bearing load (computed axis deflection equals center span deflection plus one-half vertical diameter deformation); and vertical diameter deformation against center bearing load. Figure 17 shows these curves for 30-in. Class 125, and Fig. 18 for 36-in. Class 100, prestressed concrete cylinder pipe (14-ft. beam span).

TABLE 3

Beam Test Specimen Data

Prestressed Pipe		Design Pressure *psi.*	Bearing Block Width—*in.*	
Size *in.*	Description*		Center	Ends
16	[illegible] in. M.B.U. 1.0 in. C.C. P.W.	150	8	6
20	[illegible] in. M.B.U. 1.0 in. C.C. P.W.	100	8	6
24	[illegible] in. M.B.U. 0.925 in. C.C. P.W.	100	10	8
30	6 ga. M.B.U. 1.00 in. C.C. P.W.	125	12	10
36	6 ga. M.B.U. 0.857 in. C.C. P.W.	100	12	12

* All No. 16 ga. cylinders.

Conclusion

Prestressed concrete steel cylinder pipe has been extensively tested in the laboratory. The excellent service record of this pipe provides an even greater assurance that conservative principles have been employed in its design.

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A River Undercrossing at Peterborough

By Ross L. Dobbin

A paper presented on April 3, 1950, at the Canadian Section Meeting, Niagara Falls, Ont., by Ross L. Dobbin, Gen. Mgr., Utilities Com., Peterborough, Ont.

PETERBOROUGH, Ont., is divided into two parts by the Otonabee River, and approximately one-quarter of the population is in the eastern portion, known as "East City." The pumping station and supply mains are on the west side of the river. Prior to 1949 East City was supplied by two 8-in. mains, which had been sufficient until about 1940, when there began to be complaints of low pressures in the higher parts of the district. As the population increased, these conditions became much worse.

FIG. 1. Welding Pipe at Assembly Area

Immediately after the war's close the city began to expand rapidly, and the water department was too busy extending distribution facilities to spend time or money on improvements to service. In 1948, however, it was decided to install another supply main to East City, and various routes were considered. A dredge of a size that would be able to get through the locks on the Trent Canal was not available at the time, and to do the trenching across the river by divers would have added very considerably to the cost. Therefore, the project was abandoned for the time being, but early in 1949 the city council decided to connect the sewerage system in East City with the system in the main part of the city, which would necessitate laying a sewer across the river. Advantage

was taken of this situation, and a proposal was made by the utilities commission to lay a sewer main, as well as a water main, in the same trench at the same time. The city council agreed to pay one-half the cost of the river crossing.

It was discovered that a small clam shell dredge belonging to the McNamara Construction Co. would be available on a rental basis to excavate the necessary trench. The required quantity of steel pipe was ordered at once, steel being preferred to cast iron because it could be welded on the shore and installed as a unit.

Fig. 2. Pipe Launching

The site selected was one where a short peninsula juts out from each bank of the river, at which point it had been decided to lay the sewer main. The river there was only about 300 ft. wide, and the property at both ends belonged to the city.

The 12-in. steel pipe was lap welded, weighed 45 lb. per lineal foot and was tested to 1,000 psi. at the factory. The ends were beveled for welding. This pipe was delivered in June 1949 and piled near the site. The dredge and landing barge arrived on July 22 and started to work on July 25.

The excavation for the two pipes was 7 ft. wide, level from bank to bank and deep enough to allow 1 ft. of cover over the pipe in the middle of the river. It was carried into the banks for a distance of about 30 ft. The hardpan excavated material was wasted below the site, but as much as possible was stockpiled for backfilling.

The pipe was assembled in two lines on the bank of the river and parallel to it, resting on 12 × 12-in. stop logs. It was lined up and then welded, this work being done by a local welder who used two passes of Lincoln No. 5 rod. The arrangement, shown in Fig. 1, permitted the pipe to be rolled so that the welder could work down at all times. The total length of each line after welding was 294 ft., and the ends were closed by steel welding flanges with blank plates bolted on. The pipes were tested with water to 150 psi., which was twice the normal working pressure. Each joint was tested under pressure by

striking with a 4-lb. sledgehammer. There were no leaks. The two lines of pipe were then tied together, using short pieces of $1\frac{1}{2} \times 1\frac{1}{2}$-in. angles, tacked at 4-ft. centers, in alternate 20-ft. panels. This was done so that the two pipes could be placed in one operation (*see* Fig. 3).

Before any welding, the pipes were cleaned with wire brushes to remove all scale and dirt, and were spray painted inside and out with C.I.L. T2302 paint as a primer coat and C.I.L. V476 as an over coat. After welding, all joints were touched up with these paints by hand. Cast-iron 90-deg. elbows were then bolted to the ends of the pipelines and the blank flanges replaced. The latter were drilled to receive $\frac{3}{4}$-in. iron pipe nipples and valves so that water could be introduced to sink the pipes when they were in final position.

When everything was ready, on August 17, the dredge was anchored in the river opposite the center of the pipes, and lines operated by winches on the dredge were run to the ends and middle. Additional stop logs were placed for the pipes to slide on, and, at a given signal, the pipes were launched (Fig. 2). Much to the surprise of the thousand or more spectators, the pipes floated, with about 4 in. of the diameter showing. This confirmed the calculations made in the spring, when the method of laying was determined on.

Before the pipes were pulled into position, guide piles were driven at 30-ft. centers on the downstream side of the trench, with a twin pair at each end. The pipes were pulled across by a man in a row boat and tied in position to the guide piles with block and tackle to facilitate lowering. Figure 3 shows the pipes floating in position, and the method of cross bracing can be seen.

The blank flanges were then removed and the 9-ft.-long steel riser pipes bolted on. Figure 4 shows this work in progress. The scaffolding, which supported the ends of the pipes while the risers were being placed, also aided in the sinking operation. With the risers in place, the blank flanges were bolted on again, and, by means of a small pump, river water was pumped into one end of each line of pipe while the air was let out of the other end. After two hours the pipes were bedded down in the trench, and the next morning a diver was lowered to inspect the job. He reported that the pipes were resting in good shape on the bottom.

FIG. 3. Pipes Floating in Position

The river is badly polluted where the crossing was constructed. As the pipes were filled with this water, great precautions had to be taken to see that the main was thoroughly cleaned before being connected to the system. This procedure took the better part of a month. First, the pipe was flushed for

two hours with city water and allowed to stand for 24 hours. Next, it was flushed again with city water. Then the pipe was filled with a heavily chlorinated water which was pumped into it at one end. Five pounds of HTH * was used. After the water had been allowed to stand for 24 hours, samples were taken for bacteriological examination. The samples showed pollution in about eighteen hours. The pipe was flushed out again with city water, refilled with heavily chlorinated water and allowed to stand for 24 hours. Since samples showed pollution, the process was repeated. After two more repetitions the samples showed no pollution, so that it was safe to connect the crossing to the system. It has been in use ever since.

Backfilling was done by the clam on the dredge, except for the shore ends, which were filled by a bulldozer. Some gravel was used to backfill the middle portion.

The cost of the crossing for two 294-ft. lines of 12-in. pipe was as follows:

Pipe and fittings	$ 3,800.00
Rent of dredge and moving to site	6,700.00
Labor	650.00
Welding	530.00
Painting	260.00
Miscellaneous materials	1,041.44
Total	$12,981.44

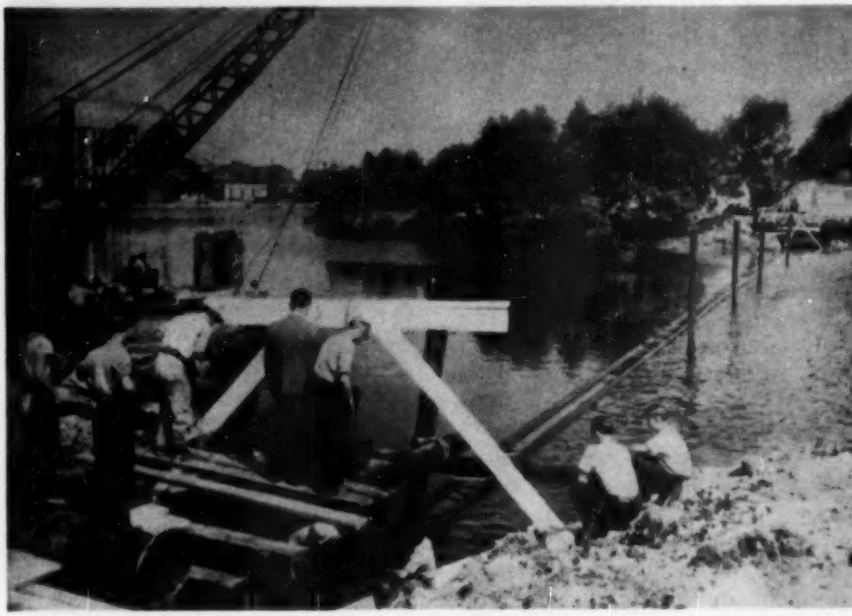

Fig. 4. Placing of Risers

If only one line of pipe had been laid, there would have been a saving of $2,500. The average cost for one pipe would have been about $35 per foot installed.

Credit should be given to Gore and Storrie, consulting engineers for the city sewerage system, and to the McNamara Construction Co., for advice and help.

* A product of Mathieson Chemical Corp., Baltimore, Md.

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1950

American Water Works Association

Tentative
STANDARD SPECIFICATIONS
for
AMMONIUM SULFATE

These "Tentative Standard Specifications for Ammonium Sulfate" are based upon the best known experience and are intended for use under normal conditions. They are not designed for use under all conditions and the advisability of use of the material herein specified in any water treatment plant must be subjected to review by the chemist/engineer responsible for operations in the locality concerned.

Approved as Tentative by the Board of Directors of the A.W.W.A.
on July 15, 1949

First Printing, November 1950

AMERICAN WATER WORKS ASSOCIATION
Incorporated
500 Fifth Avenue, New York 18, N.Y.

Table of Contents

These "Tentative Standard Specifications for Ammonium Sulfate" were prepared under the direction of C. K. Calvert (deceased) and J. E. Kerslake of the Water Purification Division, A.W.W.A. The specifications were approved by the Executive Committee of the Water Purification Division and by the Water Works Practice Committee, and received the approval of the Association's Board of Directors on July 15, 1949. The specifications were also submitted for review to producers and consumers of the materials involved, whose comments were then considered by Norton A. Thomas, acting as referee. The text as finally edited was approved by the Executive Committee of the Water Purification Division on July 10, 1950.

Tentative
Standard Specifications for Ammonium Sulfate

Part A—Material Specifications

Sec. 1A—Scope

These specifications cover ammonium sulfate for use in the treatment of municipal and industrial water supplies. The specifications are intended for use in connection with Part B (Sampling, Inspection, Packing and Marking) and Part C (Testing Methods) of this document.

Sec. 2A—Definition

The commercial grade of ammonium sulfate, suitable for use in water treatment in connection with chlorination, consists of rhombic crystals or fine granules and is of a grayish-white to brown color, depending on the extent of its purification. The color is due to traces of iron and organic compounds having their origin in the source of the material, which is usually a by-product in the manufacture of coal gas and coke.

Sec. 3A—Sampling

Sampling shall be conducted in accordance with Part B (Sampling, Inspection, Packing and Marking) of this document.

Sec. 4A—Methods of Testing

The laboratory examination shall be carried on in accordance with Part C (Testing Methods) of this document.

Sec. 5A—Impurities

The ammonium sulfate supplied under these specifications shall contain no soluble mineral or organic substances in quantities capable of producing deleterious or injurious effects upon the health of those consuming the water which has been treated properly with the ammonium sulfate.

Sec. 6A—Rejection

6A.1. Notice of dissatisfaction with a shipment, based on the specifications, must be in the hands of the consignor within ten days after receipt of the shipment at the point of destination. If the consignor desires a retest, he shall notify the consignee within five days of notice of the complaint. Upon receipt of the request for a retest, the consignee shall forward to the consignor one of the sealed samples. In the event that the results obtained by the consignor, on retesting, do not agree with the results obtained by the consignee, the other sealed sample shall be forwarded, unopened, for analysis to a laboratory agreed upon by both parties. The results of the referee analysis shall be accepted as final and the cost of the referee analysis shall be paid for by the party whose results show the greatest discrepancy from the referee results.

6A.2. On the basis of the retest or the referee test, the consignor may remove the material from the premises of the consignee or a price adjustment may be agreed upon by the consignor and consignee.

Sec. 7A—Physical Requirements

The material shall be homogeneous and in a fine crystalline form, suitable for feeding with dry feeders. It shall not lump extensively in storage and shall be free from lint, chips or other foreign matter.

NOTE: It has been found that the standard ammonium sulfate which meets these specifications will cake and arch when stored in bulk. To prevent this, the purchaser may specify the addition of a small amount of finely ground stucco (calcium sulfate) to the material.

Sec. 8A—Chemical Composition

8A.1. The moisture content shall not exceed 0.25 per cent.

8A.2. The available ammonia content, expressed as NH_3, shall be not less than 25.0 per cent.

8A.3. The ether-soluble matter shall not exceed 0.03 per cent.

8A.4. The free-acid content, expressed as H_2SO_4, shall not exceed 0.15 per cent.

8A.5. The pyridine content shall not exceed 0.05 per cent.

8A.6. The ammonium sulfate shall contain no cyanide when tested in the prescribed manner.

Sec. 9A—Taste

The material shall contain no impurity which will produce an unpleasant taste or odor in the water in the absence of chlorine or in combination with chlorine in concentrations required for treatment of a water supply.

Part B—Sampling, Inspection, Packing and Marking

Sec. 1B—Scope

These procedures for the sampling, inspection, packing, weighing and marking of ammonium sulfate are intended for use in connection with Part A (Material Specifications) and Part C (Testing Methods) of this document.

Sec. 2B—Sampling

2B.1. Samples shall be taken at the point of destination.

2B.2. If the ammonium sulfate is handled by conveyor or elevator, a mechanical sampling arrangement may be used.

2B.3. If the material is packaged, 5 per cent of the packages shall be sampled. No sample shall be taken from a broken package.

2B.4. Ammonium sulfate may be sampled, by the use of a sampling tube which is at least ¾ in. in diameter, from carload shipments in bulk or from packages.

2B.5. The gross sample, weighing at least 10 lb., shall be mixed thoroughly and quartered to provide three 1-lb. samples. These shall be sealed in airtight, moistureproof glass containers. Each sample container shall be labeled to identify it and the label shall be signed by the sampler.

Sec. 3B—Packing and Shipping

3B.1. Ammonium sulfate may be specified to be shipped in boxes, kegs, barrels or bags (either multiwall paper or lined burlap) in sizes ranging from 25 lb. to 400 lb. net. It may also be shipped in bulk, in paper-lined boxcars.

3B.2. The net weight of packages shall not deviate from the recorded weight by more than 2.5 per cent, plus or minus. If exception is taken to the weight of the material received, it shall be based on a certified unit weight of not less than 10 per cent of the packages shipped, selected at random from the entire shipment.

Sec. 4B—Marking

Each shipment of material shall carry with it some means of identification. Each package shall have marked legibly thereon the net weight of the contents, the name of the manufacturer and a brand name, if any. The package may bear also the statement "Guaranteed by (name of manufacturer) to meet the specifications of the American Water Works Association for ammonium sulfate."

Part C—Testing Methods

Sec. 1C—Scope

These methods for the examination of ammonium sulfate are intended for use in connection with Part A (Material Specifications) and Part B (Sampling, Inspection, Packing and Marking) of this document.

Sec. 2C—Sampling

2C.1. Sampling shall be conducted in accordance with Part B (Sampling, Inspection, Packing and Marking) of this document.

2C.2. The sample delivered to the laboratory shall be quartered to approximately 100 g. After thorough mixing, this sample should be stored in an airtight glass container and weighed out of it rapidly to avoid change in moisture content.

2C.3. The laboratory examination of the sample shall be completed within five working days after receipt of the shipment.

Sec. 3C—Moisture

3C.1. *Procedure.* Weigh accurately approximately 10 g. of the sample into a short weighing bottle and dry to constant weight in an oven at 103°C.

3C.2—*Calculation:*

$$\frac{\text{Loss in weight}}{\text{Weight of sample}} \times 100 = \text{per cent moisture}$$

Sec. 4C—Available Ammonia—Formaldehyde Method

4C.1—*Reagents:*

(a) 1.0*N* sodium hydroxide, carbonate free

(b) 30 per cent solution of formaldehyde which has been neutralized exactly with sodium hydroxide using phenolphthalein indicator (this must be freshly prepared)

4C.2—*Procedure:*

4C.2.1. Dissolve 1 g. of the sample in boiled and cooled distilled water and neutralize the acidity with sodium hydroxide using methyl orange indicator. The amount of sodium hydroxide is not recorded.

4C.2.2. Add 5 ml. of formaldehyde [4C.1(b)] and shake gently a few times.

4C.2.3. Titrate with 1.0*N* sodium hydroxide using phenolphthalein indicator.

4C.3—*Calculation:*

$$\frac{\text{ml. } 1.0N \text{ NaOH} \times 0.01703}{\text{Weight of sample}} \times 100 = \text{per cent ammonia } (NH_3)$$

Sec. 5C—Available Ammonia—Alternate Method

5C.1—*Reagents:*

(a) Approximately 10*N* sodium hydroxide

(b) $1.0N$ sulfuric acid

(c) $0.1N$ sodium hydroxide

(d) Distilled water substantially ammonia free

(e) Methyl red indicator

5C.2—*Procedure:*

5C.2.1. Weigh accurately approximately 2.5 g. of the sample of ammonium sulfate and put in a Kjeldahl flask fitted with a spray trap and suitable condenser. Add 150 ml. of distilled water [5C.1(d)] and 15 ml. of sodium hydroxide solution [5C.1(a)].

5C.2.2. When the sample is completely dissolved, distill at least 100 ml. from the Kjeldahl flask into 40 ml. of $1.0N$ sulfuric acid [5C.1(b)], keeping the delivery tube just under the receiving liquid.

5C.2.3. Titrate the distillate with $0.1N$ sodium hydroxide [5C.1(c)] using methyl red indicator.

5C.3—*Calculation:*

$$\frac{[\text{ml. } 1.0N\ H_2SO_4 - (\text{ml. } 0.1N\ \text{NaOH} \times 0.1)] \times 0.01703}{\text{Weight of sample}} \times 100 = \text{per cent ammonia } (NH_3)$$

Sec. 6C—Ether-soluble Matter

6C.1. *Reagent.* Anhydrous ethyl ether.

6C.2—*Procedure:*

6C.2.1. Weigh accurately approximately 25 g. of the sample after drying for two hours at 103°C.

6C.2.2. Extract for 2 hours in a Soxhlet extractor.

6C.2.3. Dry the residue in a moisture-free atmosphere to constant weight.

6C.3—*Calculation:*

$$\frac{\text{Weight of ether-soluble residue}}{\text{Weight of sample as received}} \times 100 = \text{per cent ether-soluble matter}$$

To obtain the weight of the sample as received, use the equation:

$$\frac{\text{Weight of sample as weighed}}{100 - (\%\ \text{moisture})} \times 100 = \text{weight of sample as received}$$

Sec. 7C—Free Acid

7C.1—*Reagents:*

(a) $0.1N$ sodium hydroxide

(b) Methyl red indicator

7C.2—*Procedure:*

7C.2.1. Dissolve 24.5 g. of the sample in 100 ml. of distilled water and filter into a 500-ml. flask.

7C.2.2. Rinse the beaker and wash the filter with not more than 50 ml. of water.

7C.2.3. Add four drops of methyl red indicator and if the solution has a reddish tint, titrate with $0.1N$ NaOH, added dropwise, with constant shaking, until a yellowish tint develops.

7C.3—*Calculation:*

$$\text{ml. } 0.1N\ \text{NaOH} \times 0.02 = \text{per cent free acid (as } H_2SO_4)$$

Sec. 8C—Pyridine

8C.1—*Reagents:*

(a) Approximately $1.0N$ sodium hydroxide

(b) Approximately $1.0N$ sulfuric acid

(c) $0.1N$ sodium hydroxide

(d) $0.1N$ sulfuric acid

(e) Sodium hypobromite solution (dissolve 100 g. of NaOH in 800 ml. of distilled water and cool; add 25 ml. of liquid bromine and shake until all is dissolved; make up to 1 liter; this solution must be prepared just prior to time of use)

NOTE: Liquid bromine volatilizes readily and the vapor is violently corrosive and extremely irritating to the mucous membrane upon inhalation. If the liquid comes in contact with the flesh, it produces severe burns which

are difficult to heal. When working with this element in the laboratory, the analyst should use a well ventilated hood and take any other protective measures necessary.

(f) Methyl orange indicator

8C.2. *Apparatus*. Ammonia distillation apparatus.

8C.3—*Procedure:*

8C.3.1. Dissolve 50 g. of the sample in 150 ml. of distilled water in a 1-liter distilling flask and add two drops of methyl orange indicator. Add 1.0N NaOH [8C.1(a)] to the endpoint and 5 ml. in excess.

8C.3.2. Distill for 30 minutes, using glass beads, into a 1-liter flask containing 10 ml. of 1.0N H_2SO_4 [8C.1(b)] in 100 ml. of distilled water. It is important to keep the condenser cool.

8C.3.3. To the cooled distillate, add about 110 ml. of sodium hypobromite solution [8C.1(e)] and shake well.

8C.3.4. Distill as much as possible, using glass beads, into a 500-ml. flask containing 10 ml. of 0.1N H_2SO_4, being careful to keep the receiver cool.

8C.3.5. Titrate with 0.1N NaOH using methyl orange indicator.

8C.4—*Calculation:*

$$\frac{\text{ml. } 0.1N\ H_2SO_4 - \text{ml. } 0.1N\ \text{NaOH} \times 0.0079}{\text{Weight of sample}} \times 100 = \text{per cent pyridine}$$

Sec. 9C—Cyanides

9C.1—*Reagents:*

(a) 50 per cent solution of sodium hydroxide

(b) 10 per cent solution of ferrous sulfate

(c) 10 per cent solution of ferric chloride

(d) 1 : 3 sulfuric acid, by volume

9C.2—*Procedure:*

9C.2.1. Dissolve 2 g. of the sample in a minimum amount of water in a test tube and then add a few drops of sodium hydroxide solution [9C.1(a)].

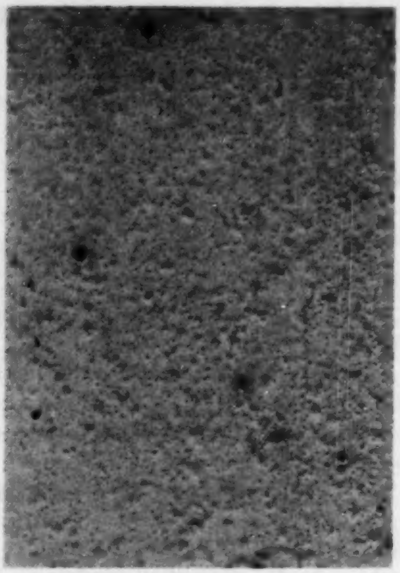
9C.2.2. Add one drop of ferrous sulfate solution [9C.1(b)] and one drop of ferric chloride solution [9C.1(c)] and shake.

9C.2.3. Add 3 ml. of 1 : 3 sulfuric acid [9C.1(d)]. The presence of cyanide is shown by the development of a blue color.

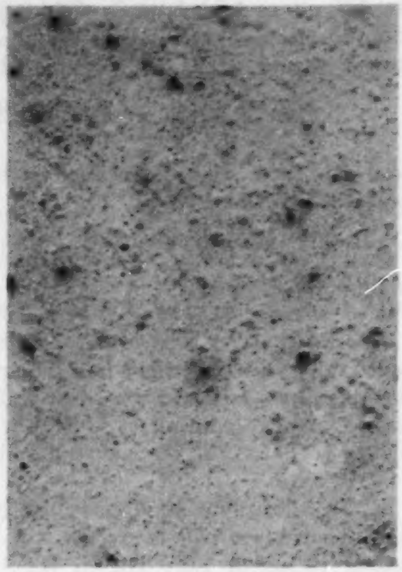
Breakdown of Paint Films on Steel Surfaces

The photographs on the following page are taken from a report issued by the British Iron and Steel Research Assn. in December 1949. According to the report, these photographs, "which are at natural size, depict different grades of breakdown, by rusting, of painted steel surfaces exposed in the tests of Joint Technical Panel J/Pl—Paints for Structural Steelwork—of the [British Iron and Steel Research Assn.] Protective Coatings Sub-Committee. In the opinion of the Panel, under practical conditions repainting would be desirable when between 0.2 per cent and 0.5 per cent rusting has been reached."

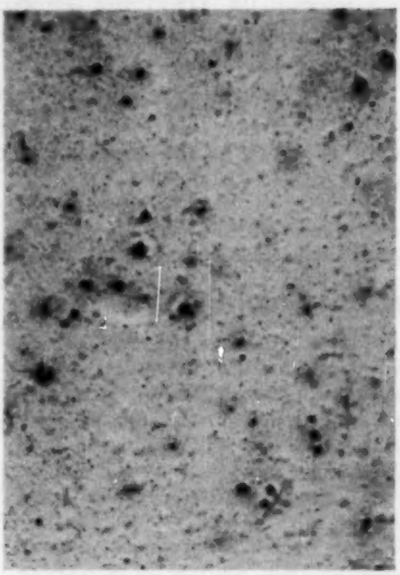
The report cautions that "these paint films were applied to specimens prepared for painting by weathering and wire-brushing and that the type of breakdown would be different in cases where the priming paint had been applied over a freshly descaled and unrusted surface."



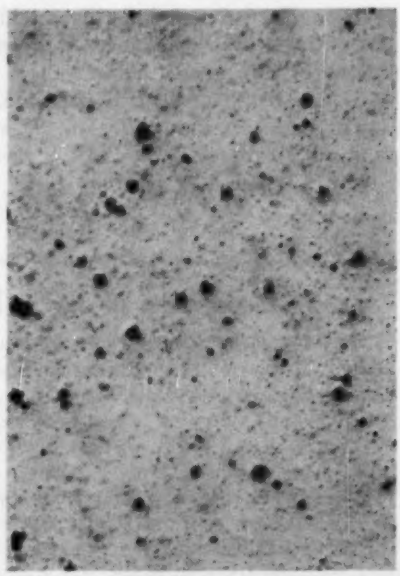
0.1 Per Cent Rusting



0.2 Per Cent Rusting



0.5 Per Cent Rusting



1.0 Per Cent Rusting

Degrees of Rusting of Painted Steel Surfaces

(See preceding page for explanation.)

Faust to Join A.W.W.A. Staff

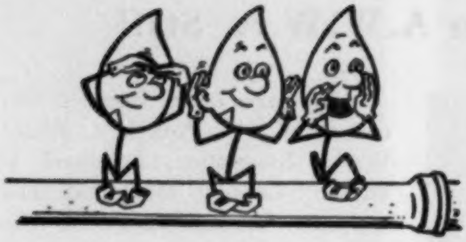
By action of the Board of Directors of the American Water Works Association, Raymond J. Faust of Lansing, Mich., has been appointed Assistant Executive Secretary of the American Water Works Association, effective on or about January 1, 1951.

Faust at the present time is Chief of the Water Supply Section, Division of Engineering, Michigan Department of Health. He was born in Millersburg, Pa., September 16, 1901. He was graduated from Pennsylvania State College with the degree of B.S. in Sanitary Engineering in 1923 and received his C.E. from the same institution in 1935.

From the time of his graduation in 1923, Faust has been in the continuous employ of the Michigan Department of Health, Division of Engineering. His duties covered the entire gamut of public health engineering in the water and sewage fields. In recent years his duties have been confined principally to water supply, and his responsibilities in administrative and promotional matters have largely replaced his field activities.

Faust is at present a member of the Board of Directors of the American Water Works Association, representing the Michigan Section. He joined the Association in January 1938 and has been active in the affairs of the Section since that time, serving the Section as trustee in 1939–41 and as Secretary-Treasurer from 1944 until the time of his election as Director in 1949. He received the Fuller Award in 1946.

Faust is a registered professional engineer in Michigan and is a member of the American Public Health Association and the Lansing Engineers Club.



Percolation and Runoff

Engineers Joint Council Report: When, on January 3, 1950, President Truman appointed the Water Resources Policy Commission, he gave official recognition at the highest level to the complexity of problems which have derived from the water needs of rapidly growing cities and industries; from the tremendous demand for water created by its increased use in irrigation; and from the conflicting concepts of water resources control and development manifested by the activities of the Corps of Engineers and the Bureau of Reclamation.

The able commission appointed by the President was given until December 1, 1950, to submit its report to him. The commission, recognizing the necessity of developing information from every available source, addressed the governors and attorneys general of the states, and more than 800 private and public agencies, requesting suggestions concerning the nature of the problem and the steps that might be taken to correct it.

The most conspicuous return made to the commission was by a special committee of the Engineers Joint Council on July 1, 1950. This brochure brought together special reports made by more than one hundred engineers organized into nine task groups. These special reports were epitomized by the Coordinating Committee which worked intensively under the leadership of Abel Wolman. The American Water Works Association has profited by the services of Dr. Wolman, acting for a number of years as the chairman of its Committee on National Water Policy. The last report made by that committee was published as an official document of A.W.W.A. in the Journal for July 1948.

The Engineers Joint Council "Statement of Desirable Policy With Respect to the Conservation, Development and Use of the National Water Resources" is the engineers' formula, effectively expressed, for the solution of a serious national problem, and merits the serious consideration of every person who may have an enlightened interest in water resources development in the United States.

(*Continued on page 4*)

(Continued from page 2)

Water in Korea became the subject of heated Congressional debate the other day after the Pentagon had been bullied into banning free beer for men at the front and then quickly uproared into unbanning it again. With Rep. Joseph R. Bryson (Dem., S.C.) trying to push through a bill to outlaw all alcoholic beverages for everybody in times of emergency and Rep. John D. Dingell (Dem., Mich.) offering a bill to provide free beer for the troops, the battle was joined. Joining Mr. Bryson, in fact, was none other than Mrs. D. Leigh Colvin, president of the WCTU, who had earlier called for action "to protect drafted youngsters against alcoholism" and was backing Bryson with a demand for a permanent emergency. As for friend Dingell, he had no end of silent partners who all but wore their mental palms out applauding, while he dramatically declared: "Water in Korea is deadlier than bullets. To force American troops to drink dangerously polluted water is the equivalent of an attack from the rear by infiltration," subjecting them to such horrors as "typhus . . . cholera . . . dysentery."

Passionately impartial ourself, we could understand how John's language got loused up on the almost unpronounceable "typhoid" and write off his heavy hyperbole to oratorical license. As for Mr. Bryson, did he really believe that he could enlist our sympathies with his left-tonsil plea for water when with his right tonsil he was vilifying the same stuff only very slightly disguised? As a matter of fact, if we remember 3.2 brew correctly, and the G.I. howl for it was really heard from Seoul to Washington, Korean water must really be bad.

Meanwhile, behind all the Congressional foam and fury, the war in Korea continued unabated and the troops in Korea continued, after only a momentary interruption, to get their free beer. Only now it is to be paid for out of Post Exchange profits instead of with taxpayers' money—one of those nice differences explainable only in Washingtonese.

And so that Mrs. Colvin won't divert her energies to worrying about what could be a really serious problem, we can report that water purification equipment is hard at work in Korea too.

(Continued on page 6)

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(*Continued from page 4*)

A modern fountain of youth is that installed in the new Canandaigua, N.Y., elementary school building. Certainly it has assured grade-scholar Judy Hamlin immortality in our eyes. One of a number of pupils asked by their principal for their impressions of the new building, it was Judy—our Judy—who answered: "I like the drinking fountain best. The water is better than any I ever tasted." As a matter of fact, we'll extend our bestowal of immortality to include *Associated Press* who reported the incident just like that. After all, it took almost as much discrimination on *AP*'s part as on Judy's to appreciate her appreciation.

And by the way, the water superintendent at Canandaigua certainly deserves a bow—even if his name should turn out to be Hamlin.

Alfred O. Norris has been appointed executive vice-president of the Indianapolis Water Co. He will continue to serve as general manager for the company, the post he assumed when he came to Indianapolis from Birmingham, Ala. (see April 1949 P&R, p. 16). Another change in personnel is the appointment of Lewis S. Finch, the company's chief engineer, as vice-president in charge of operations and engineering.

(*Continued on page 8*)

(Continued from page 6)

Four new pH standards have been selected by chemists of the National Bureau of Standards to provide fixed points at the upper and lower ends of the standard pH scale. The new solutions extend the accuracy of standard scales which are made to conform with the three middle-range standards (pH 4, 7 and 9) now distributed by the bureau to chemists. The new standard solutions fix pH values at approximately 10 and 11.7 at the high end, and 3.6 and 2.1 at the low end. Distribution of the latter two solutions is expected when an adequate supply of purified materials is available.

Scott B. Ritchie has been appointed vice-president in charge of operations of the U.S. Pipe and Foundry Co.

Kimball Blanchard, formerly with Crane Co., has been appointed sales engineer in the New York District for Rensselaer Valve Co.

The red issue swung the election in little Madison, Miss., last month, sweeping all but one member of the regular Democratic slate out of office by defeat in the conclusive primaries. More a matter of marks than Marx, however, the red in question actually issued from the water supply in the form of an unknown chemical constituent which caused staining of clothing and skin. When city officials ignored complaints from literally red-faced housewives, they overlooked some long-suffering suffrage. And when the ballot boxes were stuffed, it was a scorned woman in every post but that of city marshal. What the marshal's plan is for the time when his new bosses take office is not at all certain, but we'll venture to guess he resorts to a little redeye every time he thinks about it.

Meanwhile, vested interests in other communities which have trouble with iron and manganese staining might well take heed. Who knows when some little lady will decide to lower the boom and carry away not only your vest but those pants you claim to wear. Besides, who would want the job of First Man in the White House?

(Continued on page 10)

(Continued from page 8)

Pani Maharaj is the Hindustani expression meaning "Water King"—and that's the title which has been bestowed upon one Jeevram Vyas, 50-year-old yogi, who recently took a job on the Rajasthan Underground Water Board at a modest civil service salary of 500 rupees ($105) per month and continues his tenure there at the personal pleading of India's Food and Agriculture Minister K. M. Munshi and under the advised auspices of none other than Prime Minister Jawaharlal Nehru. What has earned for Jeevram his title and his countrywide fame has been his success in "locating underground water sources with uncanny accuracy by merely closing his eyes and pointing." And the fact that he locates water in mid-desert or mid-anywhere, "sometimes by moving his hand across the map of a region and pinpointing the source of water, . . . more often, sitting in a room or traveling in a car and seeing a cloud of haziness in the depths of the earth below," has made him particularly valuable in India. Like his less auspiciously sponsored American counterparts, the yogi can also "indicate with mathematical precision not only the quantity of water to be found, but also whether it is sweet or saline."

Perhaps because Pani Maharaj "works entirely without instruments"; perhaps because Pandit Nehru (classified as "an extremely well-schooled

(Continued on page 12)

(Continued from page 10)

and fairly skeptical man") apparently accepts him; perhaps because one of the desert wells officially accredited to him produced a generous 2.88 mgd.; perhaps because the Indian government has been careful not to confirm claims by his admirers that he can also see underground oil and precious minerals; perhaps because another Yogi (named Berra) produced miracles for the New York Yankees during the past baseball season; or perhaps because we're just plain sick and tired of our own antirhabdomantic rantings, we're ready to give up and take into at least the vestibule of our fold not only Jeevram but also Kenneth Roberts' pet dowser who had the temerity recently to find ground water in Bermuda (see June P&R, p. 6). Besides, having read last month's JOURNAL panel on scientific "Prospecting for Ground Water," we aren't at all sure we don't understand—and therefore appreciate—mysticism much better.

At the International Engineering Exhibition to be held in India during January of 1951 there will take place the Fourth Congress on Large Dams, the meeting of the International Assn. of Hydraulic Research, and several other engineering conferences. Information for exhibitors or visitors may be obtained from the Consulate General of India, 3 E. 64th St., New York 21, N.Y.

(Continued on page 14)


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(Continued from page 12)

The legal complexities surrounding West Virginia's participation in the Ohio River Valley Water Sanitation Commission are to be reviewed by the U.S. Supreme Court; and, because of the significant bearing the case has on all similar interstate compacts, the decision will be awaited with unusual interest. Background of the action is the refusal of Edgar B. Sims, state auditor, to pay West Virginia's share of the commission's operating costs, on the grounds that the Ohio River compact violated the state's constitution. His withholding of funds was upheld by the West Virginia courts, and the case has now been appealed to the highest court of the land.

At issue is the determination of whether the power of a state to enter into congressionally approved compacts with other states, in accordance with Article I of the U.S. Constitution, can be limited by provisions of the state's own constitution. Contingent questions which may also be settled are: does the constitution of West Virginia actually restrict its power to enter into interstate compacts? if so, does the Ohio River compact subject West Virginia to any obligation in violation of its constitution? and, did ratification of the compact by the West Virginia Legislature result in an unconstitutional delegation of police power or legislative authority?

(Continued on page 16)

(*Continued from page 14*)

New Jersey's utility antistrike law, as a means of effecting peaceful settlement of labor-management disputes, has received what many observers consider a major setback in a recent decision of the State Supreme Court affecting 11,000 employees of New Jersey Bell Telephone Co. The court set aside an arbitration award reached under the terms of the antistrike law, ruling that the board had not properly applied the standards set up in the law as guides in determining wage increases, and that in any event the board could not make an arbitration award of a union shop because the Taft-Hartley law requires that this type of union security result from an agreement of company and union.

Both AFL and CIO union leaders—never very happy about the antistrike law before, although they have used its arbitration provisions repeatedly—now claim that successful arbitration under the law is ended. Representatives of a transit workers' union have already declared that unless the law or the court decision is changed, they will not continue to abide by the antistrike statute when their contract expires. The unions also point out that, under the antistrike law, deprivation of their major bargaining threat has resulted in prolonging labor disputes and forcing arbitration as a substitute for free collective bargaining. They cite a settlement by bargaining—also with New Jersey Bell—which was only reached after 18 months' delay, and claim that the issue could have been settled speedily if management had not been able to count on the antistrike law to remove the threat of drastic union reaction to uncompromising unsatisfactory proposals.

Governor Driscoll is reported as being disturbed by recent developments and has indicated that the state might request further court review of the power of arbitration boards to settle the union shop issue. Labor resentment is running so high, however, that a concerted drive to repeal the antistrike law when the legislature convenes in January is a certainty, regardless of the governor's actions.

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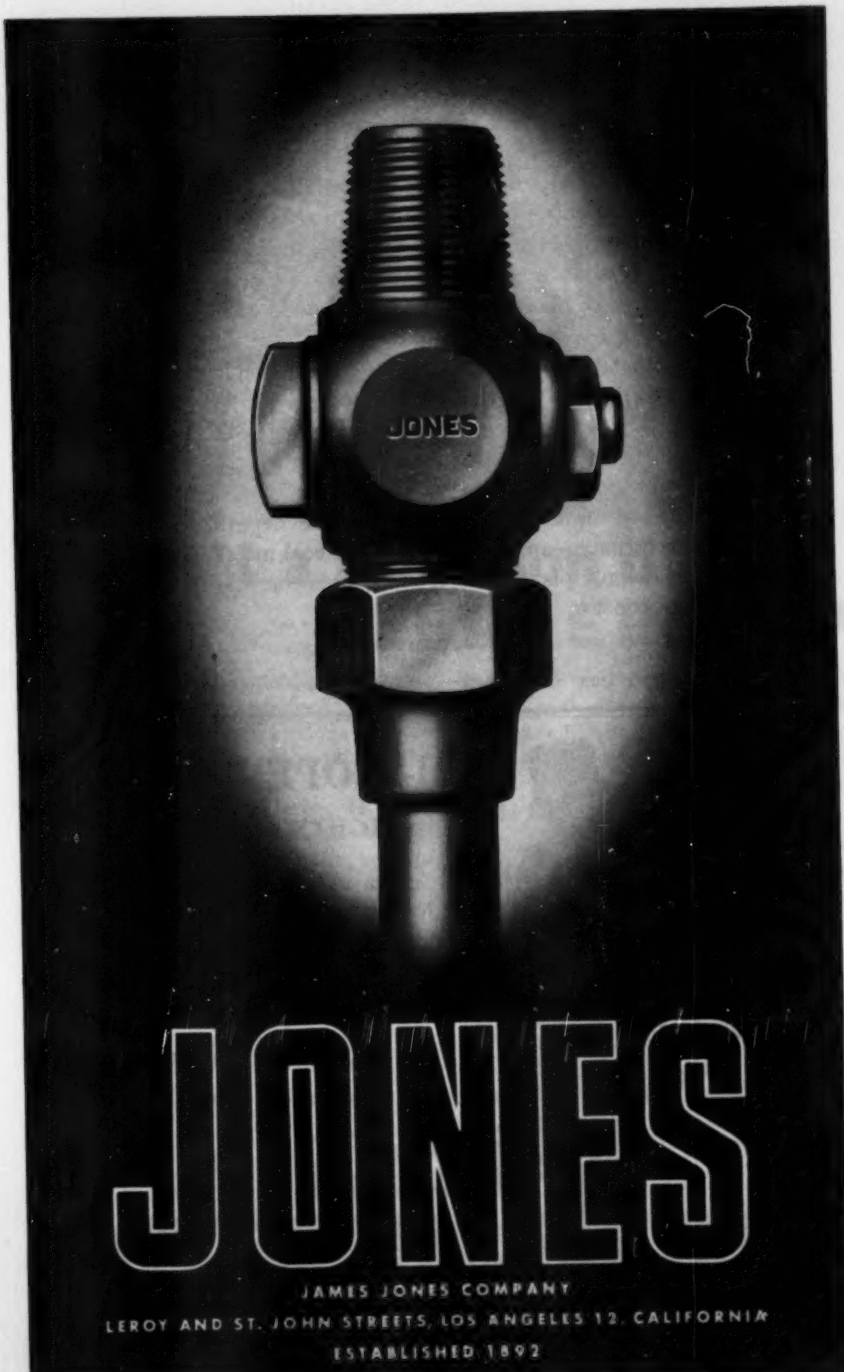
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(Continued from page 16)

Shortages of manpower, which are beginning to share the headlines with developments in Korea, have elicited a reminder from the Veterans Administration of an excellent source of trainees in the thousands of disabled veterans who wish to take advantage of the Vocational Rehabilitation Act. According to VA, surveys have shown that handicapped workers, when properly placed, prove to be at least as good as their able-bodied fellow employees. There are even indications that they suffer fewer accidents and produce better than others. Full details on placement under the rehabilitation act may be obtained from any VA field office.

A new $1,500,000 pipe foundry has been completed by Lone Star Steel Co. at Lone Star, Tex., and is producing 18-ft. lengths of cast-iron pressure pipe in 3- to 12-in. sizes by the centrifugal De Lavaud process. Production is at the rate of 80,000 tons of pipe annually. Additional equipment is on order which will permit the casting of pipe up to 20 in. in diameter. The company mines its own open-pit ore deposits—located within sight of its blast furnace—and operates its own coal mines in the Southwest. It is said to be the only plant of its type between the Mississippi River and the Rocky Mountains.

(Continued on page 20)

(Continued from page 18)

That barrel again—that self-filling barrel in the Missouri Ozarks—is back in action. Last month when we reported (P&R, p. 1), first, that the bottomlessness of the barrel was a "genuine supernatural happening" and then, last, that its eventual overturn proved that it was the water rather than the barrel that was supersomethingorother, we didn't expect to have to drink our words. But there it is; the barrel has been righted again and is functioning in full glory, slaking the thirst of John Orr's 250 chickens with nary a dip in level. Except now the curious, whose number had reportedly reached 20,000 before the tipster showed up, are barred from the farm and it will be viewed by invitation only.

You know, it might have been more scientific the second time to fill it with beer.

Burton S. Grant has been appointed chief engineer of water works and deputy general manager of the Los Angeles Dept. of Water & Power. Formerly assistant chief engineer of water works, he has been with the department for 25 years, and now succeeds Laurance E. Goit, who has requested less arduous duties as head of the Aqueduct Div. because of his health. Grant has also been head of the Aqueduct Div., charged with the

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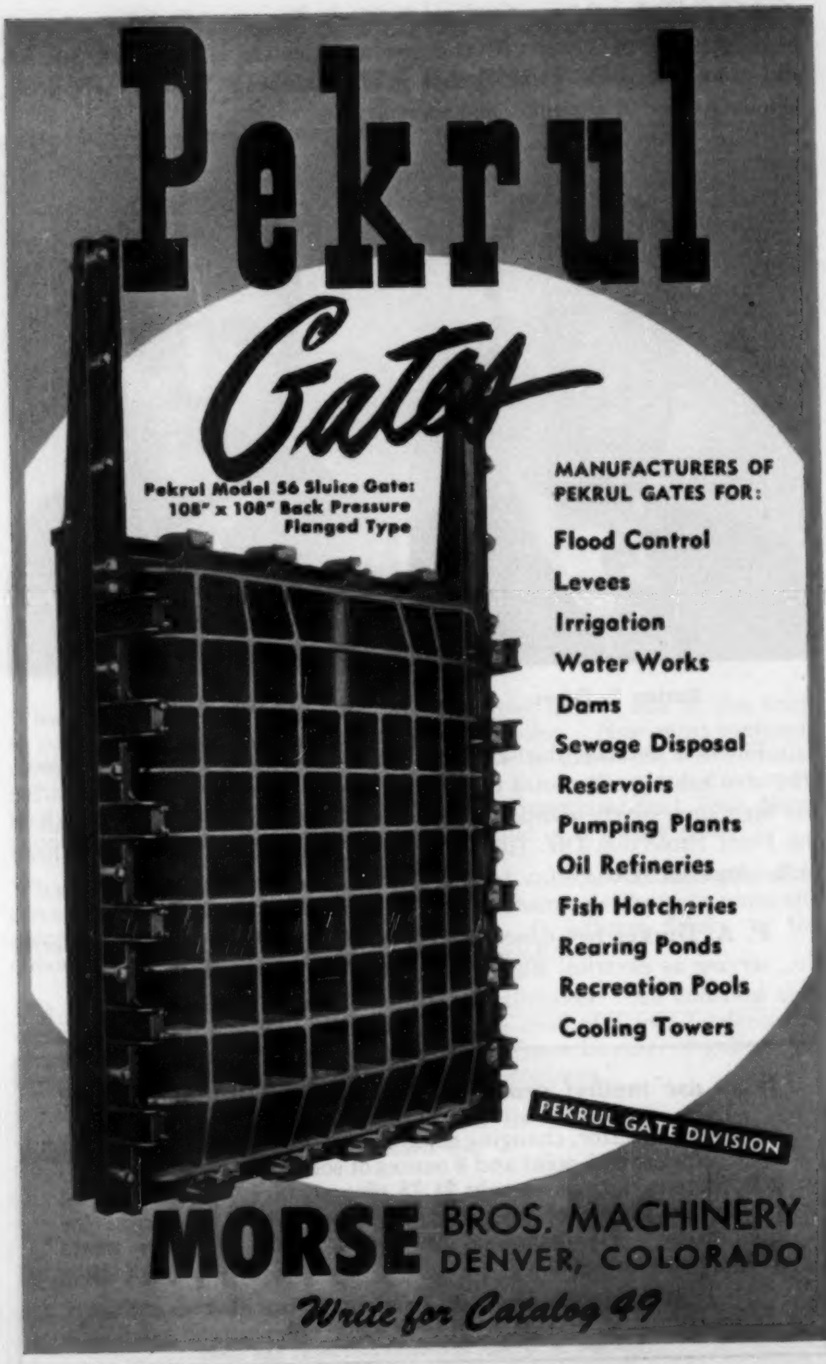
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(*Continued from page 20*)

maintenance of the Owens River Aqueduct from the High Sierra, and has held numerous posts of importance in the California Section A.W.W.A., including those of chairman and secretary.

Burton S. Grant

Samuel B. Nelson

The new assistant chief engineer of water works is Samuel B. Nelson, who also has a quarter of a century of service with the department. He has been successively assistant to the head of the Field Eng. Div., head of the Plant Protection Div. (during the war years), and, since 1949, head of the Aqueduct Div.

P. A. Durkee has joined the engineering staff of Automatic Control Co., serving as electrical engineer in the liquid level control division.

Professional Services

Membership Changes

NEW MEMBERS

Applications received September 1 to 30, 1950

Abrams, Philip, San. Eng. Asst., Dept. of Water & Power, 207 S. Broadway, Los Angeles, Calif. (Oct. '50)

Alward, H. W., Inc., Henry W. Alward, Pres., 161 Mt. Airy Rd., Bernardsville, N.J. (Assoc. M. Oct. '50)

Alward, Henry W., *see* Alward, H. W., Inc.

Anderson, Maynard M., Engr., Metropolitan Water Dist. of Southern California, 306 W. 3rd St., Los Angeles, Calif. (Oct. '50)

Angel, John, Chemist, Reasor-Hill Corp., Jacksonville, Ark. (Oct. '50)

Armstrong, James O., Mgr., Operations & Maint. Div., East Bay Municipal Utility Dist., 2127 Adeline St., Oakland, Calif. (Oct. '50)

Arnold, J. L., Gen. Supt. of Utilities, Water, Gas & Light Com., Albany, Ga. (Oct. '50)

Ashworth, Guy T., Sales Repr., Hays Mfg. Co., 515 Forrest, Shreveport, La. (Oct. '50)

Barnett, Glenn, Water Works Supt., Whiteland, Ind. (Oct. '50)

Bartlett, Russell D., Mgr., Waynesburg Water Co., 35 N. Washington St., Waynesburg, Pa. (Oct. '50)

Blanchard, Kimball, Sales Repr., Rensselaer Valve Co., Troy, N.Y. (Oct. '50)

Booth, Bernard, Clement, Lab. Asst., Water Supply Dept., Chesapeake & Ohio Ry. Co., Huntington, W. Va. (Oct. '50)

Bowen, Glenn D., Mgr., Oregon Water Corp., Box 229, Klamath Falls, Ore. (Oct. '50)

Bowen, Robert A., Sales Engr., Turbine Equipment Co., 29 Ridge St., Glens Falls, N.Y. (Oct. '50)

Burns, H. L., Supt., Hinds County Water Co., Box 762, Jackson, Miss. (Oct. '50)

Butler, Robert George, San. Engr. Asst., Dept. of Water & Power, 207 S. Broadway, Los Angeles, Calif. (Oct. '50)

Campbell, George, Acting Supt., Manhasset Lakeville Water Dist., 176 E. Shore Rd., Great Neck, N.Y. (Oct. '50)

Carey, James Lee, Results Engr., Water & Light Plant, Jonesboro, Ark. (Oct. '50)

Caughran, Lloyd, Asst. Plant Supt., Filtration Plant, 326 Patterson St., Neosho, Mo. (Oct. '50)

Chamberlin, Carl W., Pres., Brookside Water Co., Box 178, 121 E. Pikes Peak Ave., Colorado Springs, Colo. (Oct. '50)

Clarkson, Arthur W., Asst. Director, Div. of San. Eng., State Board of Health, 1036—8th Ave., Helena, Mon. (Oct. '50)

Conservation Foundation, The, Robert G. Snider, Director of Research, 30 E. 40th St., New York 16, N.Y. (Corp. M. Oct. '50)

Consolidated Electric Co., Carl J. Gardeen, Pres., 469 Broadway St., St. Paul 1, Minn. (Assoc. M. Oct. '50)

Crawshaw, Harris V., Sr. Engr., Metropolitan Water Dist. of Southern California, 306 W. 3rd St., Los Angeles, Calif. (Oct. '50)

Egger, Oscar O., Public Health Engr., State Board of Health, Dist. Office No. 3, Court House, Fond du Lac, Wis. (Oct. '50)

El Centro Water Dept., Clyde F. Lee, Supt. of Public Works, City Hall, El Centro, Calif. (Corp. M. Oct. '50)

English, Wayne, Local Mgr., Hoosier Water Co., Bloomfield, Ind. (Oct. '50)

Eppinger, William Henry, Asst. Engr., Metropolitan Water Dist. of Southern California, 306 W. 3rd St., Los Angeles 13, Calif. (Oct. '50)

Ferentz, Judson, *see* Pismo Beach (Calif.)

(*Continued on page 32*)

(*Continued from page 30*)

Furst, Walter A., Director of Public Utilities, Gainesville, Fla. (Oct. '50)

Gardeen, Carl J., *see* Consolidated Electric Co.

Gomez-Moreno, Bernardo, Supt., Municipal Acueducto, Bogota, Colombia (Oct. '50)

Green, Lawrence L., Gen. Foreman, Aqueduct Maint., Iron Mountain Headquarters, Metropolitan Water Dist. of Southern California, Rice, Calif. (Oct. '50)

Gribben, James M., Water Works Operator, Bethany Improvement Assn., Bethany, W. Va. (Affil. M. Oct. '50)

Hatch, Charles C., Jr., Supt., California Michigan Land & Water Co., 269 E. Rosemead Blvd., East Pasadena 10, Calif. (Oct. '50)

Hayes, John C., Supt., Water Dept., 240 W. Huntington Dr., Arcadia, Calif. (Oct. '50)

Herlihy, C. David, Sales Engr., American Pipe & Construction Co., 4635 Firestone Blvd., South Gate, Calif. (Oct. '50)

Howells, Wallace J., Personnel Officer, Dept. of Water Supply, 735 Randolph St., Detroit 26, Mich. (Oct. '50)

Jackson, Earl E., Dist. Mgr., American Pipe & Construction Co., 787—85th Ave., Oakland, Calif. (Oct. '50)

Kann, Bruce E., Salesman, Neptune Meter Co., 1238 N.W. Glisan St., Portland, Ore. (Oct. '50)

Katz, Sol J., Asst. Dist. Mgr., Koppers Co., Inc., 390—9th St., San Francisco 3, Calif. (Oct. '50)

Keith, H. L., Sales Engr., General Filter Co., c/o Layne-Northern Co., Inc., 401 S. DeLorenzi Ave., Mishawaka, Ind. (Oct. '50)

Kenmir, Russell Collingwood, Chief Engr., James M. Montgomery, Cons. Engr., 15 N. Oakland Ave., Pasadena 1, Calif. (Oct. '50)

LaGrange, City of, Ralph D. Witt, City Engr., LaGrange, Ga. (Corp. M. Oct. '50)

Lawton, Gerald W., Instructor, Civil Eng., Univ. of Wisconsin, Madison, Wis. (Oct. '50)

Lee, Clyde F., *see* El Centro (Calif.) Water Dept.

Lemieux, Henri Julien, Cons. Engr., Box 137, St. Joseph D'Alma, Que. (Oct. '50)

Levalley, McLeod, Inc., Emil A. Melnick, Vice-Pres., 215 E. Church St., Elmira, N.Y. (Assoc. M. Oct. '50)

McCaffery, Marguerite D., (Miss), Partner, Bowe, Albertson & Assocs., 110 William St., New York 7, N. Y. (Oct. '50)

McConnell, R. J., Dist. Sales Mgr., American Pipe & Construction Co., 4635 Firestone Blvd., South Gate, Calif. (Oct. '50)

Melnick, Emil A., *see* Levalley, McLeod, Inc.

Mengel, Carl Wayne, Engr., Wm. C. Olsen, Cons. Engrs., Raleigh, N.C. (July '50)

Miller, Burton Rich, Engr. & Partner, Snyder, McLellan, Miller & Watson, Hillsboro, Ind. (Oct. '50)

Moseley, W. Everett, Deputy Minister of Munic. Affairs, Dept. of Munic. Affairs, Provincial Bldg., Halifax, N.S. (Oct. '50)

Murphy, Clarence L., Supt., Water Works, 221—3rd St., N.E., Hampton, Iowa (Oct. '50)

Neal, Willie Grey, San. Engr., Southwest Branch Office, State Board of Health, Washington, Ind. (Oct. '50)

Nicolson, John S., Owner & Mgr., Culligan Soft Water Service, 10 N. 34, Billings, Mont. (Oct. '50)

Patterson, Allan B., Engr., Water Works Section, Dept. of Works, 511 Richmond St. W., Toronto, Ont. (Oct. '50)

Pismo Beach, City of, Judson Ferentz, Supervisor of Public Works, City Hall, Pismo Beach, Calif. (Mun. Sv. Sub. Oct. '50)

Plummer, C. A., Transite Pipe Mgr., Canadian Johns-Manville Co., Ltd., 199 Bay St., Toronto, Ont. (Oct. '50)

Rader, Earle M., Partner, Rader Knappen Tippetts Eng. Co., 1615 DuPont Bldg., Miami, Fla. (Oct. '50)

Randolph, Theodore Fitz, Engr., Harbert Constr. Corp., 2208—8th Ave., S., Birmingham 5, Ala. (Oct. '50)

Rankin, Carl Roy, Civil Engr., 330 S. Madison Ave., Pasadena, Calif. (Oct. '50)

(*Continued on page 34*)

(*Continued from page 32*)

Rian, John, Supt. of Gas & Water Operations, Northern Indiana Public Service Co., 500 Broadway, Gary, Ind. (Oct. '50)

Richard, Francis Philip, Chem. Engr., Southdown Sugars, Inc., Houma, La. (Oct. '50)

Rivers, William H., Jr., Acting Asst. Supt., Manhasset Lakeville Water Dist., 176 E. Shore Rd., Great Neck, N.Y. (Oct. '50)

Rodes, W. E., Asst. Dist. Mgr., American Pipe & Construction Co., 787—85th Ave., Oakland, Calif. (Oct. '50)

Ross, Henry F., Prin. Cost Clerk, Water Dept., 229 City Hall, Minneapolis, Minn. (Oct. '50)

Royal Allison Water Co., Edward Yablonski, Owner, California, Pa. (Corp. M. Oct. '50)

Sargent, Robert C., Mgr., Centriline Div., American Pipe & Construction Co., 4635 Firestone Blvd., South Gate, Calif. (Oct. '50)

Schweir, O. O., Sales Engr., Layne-Northern Co., Inc., 401 S. DeLorenzi Ave., Mishawaka, Ind. (Oct. '50)

Snider, Robert G., *see* Conservation Foundation, The

Squier, David C., Sales Repr., Darling Valve Mfg. Co., 63 Commodore Pkwy., Rochester 10, N.Y. (Oct. '50)

Stout, Thurman A., Civil Engr., West Virginia Water Service Co., 814 Peoples Bldg., Charleston, W. Va. (Oct. '50)

Sudrabin, Leon Peter, Chemical Engr., Electro Rust Proofing Corp., 1 Main St., Belleville, N.J. (Oct. '50)

Templeton, Gerald, Supt. of Water, Otisville, Mich. (Oct. '50)

Turcan, Alcee N., Jr., Hydr. Engr., Ground Water Branch, U.S. Geological Survey, Box 8516, Univ. Station, Baton Rouge, La. (Oct. '50)

Turner, Errett E., Master Mechanic, Thomaston Mills, Thomaston, Ga. (Oct. '50)

Ucros-Guzman, Ignacio, Chief Engr., Dept. of Design & Constr., Acueducto Municipal, Bogota, Colombia (Oct. '50)

Ustruck, John K., Field Engr., Fairbanks, Morse & Co., St. Paul, Minn. (Oct. '50)

Voedisch, Frederic W., Vice-Pres., Layne-Minnesota Co., 3140 Snelling Ave., Minneapolis 6, Minn. (Oct. '50)

Wagner, Fred, Sales Engr., Growers Tractor & Implement Co., 1300 U St., Sacramento, Calif. (Oct. '50)

Wark, John William, Hydr. Engr., Quality of Water Branch, U.S. Geological Survey, 510 Rudge Bldg., Lincoln, Neb. (Oct. '50)

Watson, Nelson, Jr., Snyder, McLellan, Miller & Watson, Financial & Eng. Counselors, Hillsboro, Ind. (Oct. '50)

White, Harold L., Chief Engr., American Pipe & Construction Co., 4635 Firestone Blvd., South Gate, Calif. (Oct. '50)

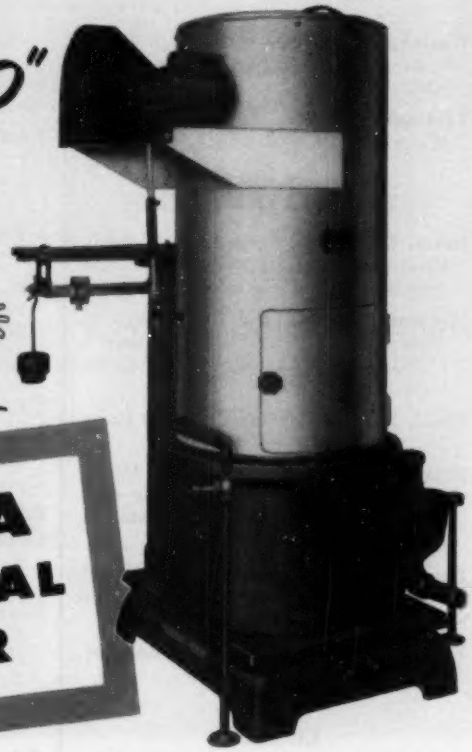
Wiesner-Rozo, Francisco, Supt., Estudios y Construcciones, Acueducto Municipal, Calle 16, Bogota, Colombia (Oct. '50)

Williams, John Davis, Watermaster & Operator, Highlands Water Co., Box 346, Clearlake-Highlands, Calif. (Oct. '50)

Witt, Ralph D., *see* LaGrange (Ga.)

(*Continued on page 36*)

IT'S THE "LAST WORD"



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(*Continued from page 34*)

Wooten, L. E., Pres., L. E. Wooten & Co., Cons. Engr., Box 2984, Raleigh, N.C. (Oct. '50)

Yablonski, Edward, *see* Royal Allison Water Co. (Pa.)

REINSTATEMENTS

Baker, Donald M., Cons. Engr., Partner, Ruscardon Engrs., 108 W. 6th St., Los Angeles 14, Calif. (Oct. '34)

McCleary, William F., Gen. Foreman, Metropolitan Water Dist. of Southern California, 306 W. 3rd St., Los Angeles, Calif. (Oct. '43)

Mead, D. H., *see* Revere Copper & Brass, Inc.

Mills, Henry J., Constr. Engr., Metropolitan Water Dist. of Southern California, 306 W. 3rd St., Los Angeles 13, Calif. (Oct. '43)

Revere Copper & Brass, Inc., D. H. Mead, Asst. Sales Mgr., 230 Park Ave., New York 17, N.Y. (Assoc. M. Aug. '28)

Vallee, Antonio, Box 187, Camaguey, Cuba (July '38)

LOSSES

Resignations

Baker, Alden K., Harrisburg Ice & Water Co., Harrisburg, Ark.

Fortson, Eugene P., Jr., Chief, Hydr. Div., Waterways Experiment Station, Vicksburg, Miss.

Glasser, C. E., Pres., General Reduction Co., 1820 Roscoe St., Chicago 13, Ill.

Hulse, C. A., Chief Sales Clerk, Canadian Johns-Manville Co., Ltd., 199 Bay St., Toronto, Ont.

Roulston, Emerson, Box 123, Dunedin, Fla.

Smith, J. F., Sales Mgr., Great Western Div., The Dow Chemical Co., 310 Sansome St., San Francisco, Calif.

Death

Gresham, Hill Campbell, Business Mgr., *Southwest Water Works Journal*, Temple, Tex. (Oct. '46)

CHANGES IN ADDRESS

Changes received between September 5 and October 5, 1950

American Concrete Pressure Pipe Assn., H. F. Peckworth, Managing Director, 228 N. LaSalle St., Chicago, Ill. (Assoc. M. Oct. '17)

Arnold, William L., Public Accountant, Box 316, San Gabriel, Calif. (July '33)

Auld, David, Deputy Director of San. Eng., Water Div., District Bldg., Washington, D.C. (Oct. '44)

Beechwood. Christian T., III, San. Engr., State Dept. of Health, 11 Gilpin Rd., Willow Grove, Pa. (Jan. '50)

Bray, Wesley R., Scranton Spring Brook Service Co., 135 Jefferson Ave., Scranton, Pa. (Jan. '48)

Burnley, Seth, *see* Staunton (Va.)

Carolina Beach, Town of, G. R. Wood, Supt. of Water & Sewer, Carolina Beach, N.C. (Corp. M. Oct. '42)

Cowser, Kenneth E., San. Engr., East Central Region, State Dept. of Public Health, 625½ S. Wright St., Champaign, Ill. (Jr. M. Jan. '49)

Crenshaw, T. B., 321 Ragan Ave., Henderson, Ky. (Oct. '47)

Culp, Russell L., 47 Mt. Auburn St., Watertown, Mass. (July '48)

Culver, Robert H., 58 Kensington Park, Arlington 74, Mass. (July '48)

Deitrick, Le Roy, Supt., Water Dept., 231 W. Mahoning St., Danville, Pa. (June '37)

Esty, Roger W., 53 Lindall St., Danvers, Mass. (Mar. '24) *Director '38–'41.*

Feiler, Alfred M., 5624—5th Ave., Pittsburgh 14, Pa. (July '48)

Gallagher, J. A., 342 South St., Corvallis, Ore. (Jan. '40)

Gloyna, Earnest F., 3706 N. Charles St., Baltimore 18, Md. (Oct. '49)

Gomez Laurens, Gilberto, 415 N. Thayer, Ann Arbor, Mich. (Jr. M. Oct. '49)

Gray, Alexander, 125 S. Main St., Fairport, N.Y. (Jan. '46)

Gulden, Horace B., Borough Mgr., Munic. Bldg., Lewistown, Pa. (Jan. '40)

Heyward, Nathaniel J., 2401 Arbuton Ave., Baltimore 30, Md. (July '45)

(*Continued on page 38*)

(Continued from page 36)

Hughes, Sydney E., Mech. Engr., 5739 Roberts Ave., Oakland 5, Calif. (Apr. '45)

Jain, Mallinath, c/o Wajrachand Jain, Osmanpura, Aurangabad, India (Jan. '50)

Jorgensen, Homer W., Cons. San. Engr., 315 Downey St., Modesto, Calif. (Oct. '39)

Kingsbury, Harold N., 1322 Chandler St., Madison 5, Wis. (Jan. '47)

Larkin, William H., Linden Ave., Prospect Heights, Rensselaer, N.Y. (Apr. '30)

Levin, Gilbert V., State Dept. of Health, 2323 Scarff St., Los Angeles 7, Calif. (Jr. M. Oct. '48)

McConnell, Alfred F., Box 26, Waco, Texas (Jan. '48)

Menzenhauer, Fred C., 554 Seminole St., Oradell, N.J. (Jan. '38)

Millikin, Joseph P., 424 Riverland Rd., S.W., Roanoke, Va. (Jan. '48)

Moye, Malcolm A., Jr., Box 1710, Tampa, Fla. (Oct. '49)

Nelson, Clarence H., Supt. of Utilities, Water & Light Dept., 106 Holmes St., Detroit Lakes, Mich. (Jan. '48)

Novaro, Joseph A., Asst. Engr., New Haven Water Co., 100 Crown St., New Haven, Conn. (Jan. '47)

Peckworth, H. F., *see* American Concrete Pressure Pipe Assn.

Peterson, C. J., 7955 Van Nuys Blvd., Van Nuys, Calif. (July '46)

Prather, J. Arthur, Local Mgr., Potomac Light & Power Co., Shepherdstown, W. Va. (Affil. Oct. '47)

Quick, Robert L., 40 N.E. 105th St., Miami 38, Fla. (Jan. '46)

Roberts, E. A., Cons. Engr., Box 573, Carlsbad, N.M. (Jan. '44)

Robinson, James M., Sales Repr., A. P. Smith Mfg. Co., 38 Main St., Hornell, N.Y. (Jan. '47)

Russell, Jess L., Director, Public Relations, Russell & Axon, 408 Olive St., St. Louis, Mo. (Oct. '47)

Schaefer, Edward J., Dist. Engr., U.S. Geological Survey, Veterans Administration Bldg., 52 Starling St., Columbus 8, Ohio (July '47)

Spiegler, Kurt S., Israel Ministry of Agriculture, Water Dept., Hakiryah, Israel (Oct. '49)

Staunton, City of, Seth Burnley, City Mgr., Staunton, Va. (Corp. M. Oct. '43)

Sterns, Edward A., Supt., Water Dept., 122 Woodview Ave., Hamburg, N.Y. (Apr. '37) *Fuller Award '38.*

Stone, J. W. L., Fischer Well & Pump Co., 711 First American State Bank Bldg., Wausau, Wis. (Jan. '49)

Stuart, Charles L., Southern California Water Co., 1206 S. Maple Ave., Los Angeles 15, Calif. (Jan. '40)

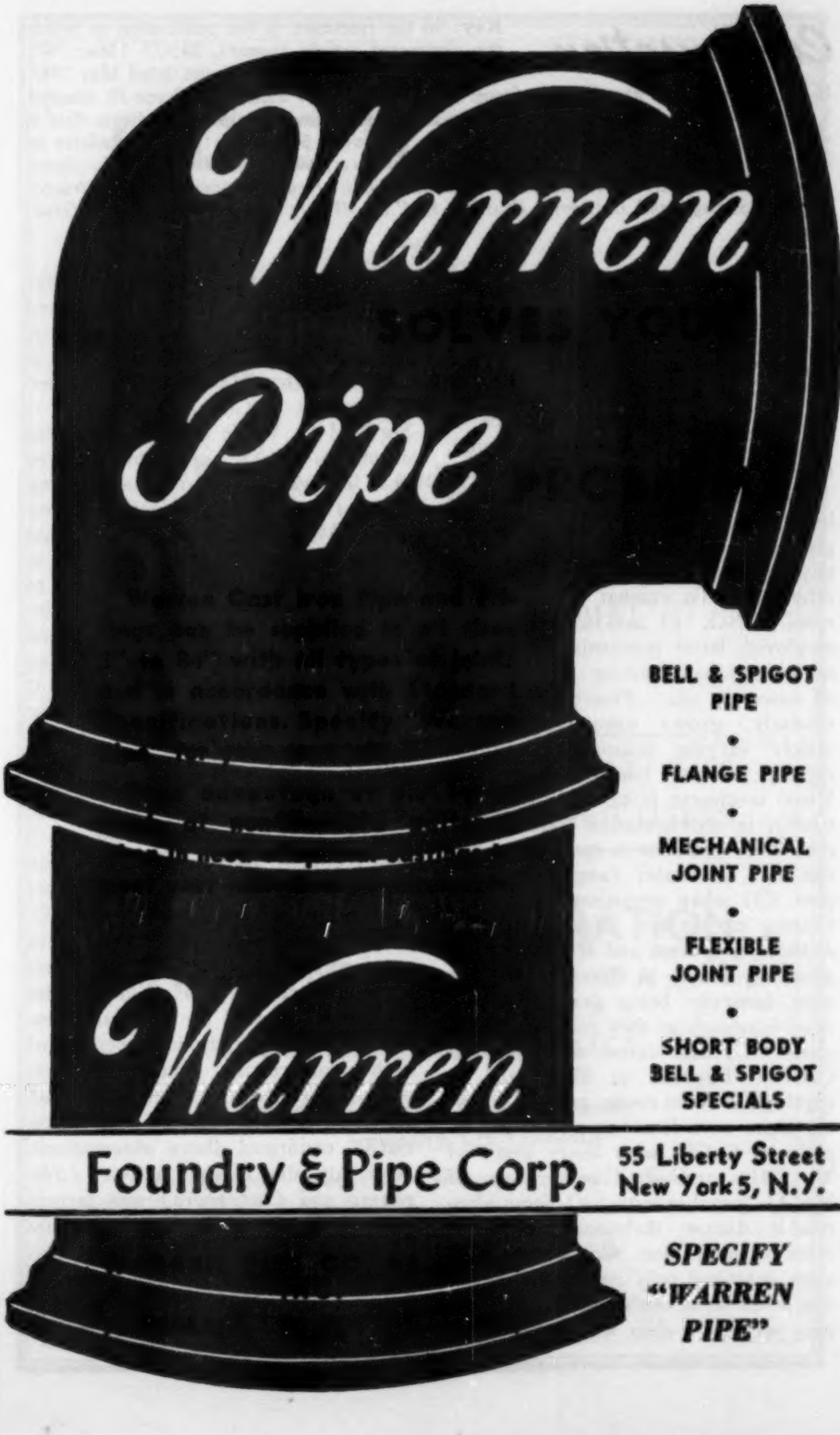
Thomas, Charles O., Citizens National Bank Bldg., Hope, Ark. (Jan. '40)

Townsend, Hal C., Repr., Worthington-Gamon Meter Co., 1975 Eager Rd., R.R. 5, Howell, Mich. (Jan. '38)

Watson, Kenneth S., Mfg. Policy Div., General Electric Co., Bldg. 36, Schenectady, N.Y. (Jan. '39)

Weed, Frederick H., 70 M Rockledge Rd., Hartsdale, N.Y. (Nov. '25)

Wood, G. R., *see* Carolina Beach (N.C.)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Key: In the reference to the publication in which the abstracted article appears, 39:473 (May '47) indicates volume 39, page 473, issue dated May 1947. If the publication is paged by the issue, 39:5:1 (May '47) indicates volume 39, number 5, page 1, issue dated May 1947. Abbreviations following an abstract indicate that it was taken, by permission, from one of the following periodicals: *B.H.—Bulletin of Hygiene (Great Britain); C.A.—Chemical Abstracts; Corr.—Corrosion; I.M.—Institute of Metals (Great Britain); P.H.E.A.—Public Health Engineering Abstracts; S.I.W.—Sewage and Industrial Wastes; W.P.R.—Water Pollution Research (Great Britain)*.

ALGAE

Effects of Algae on Water Supply. W. J. Chamberlain. Univ. Queensland Papers, Dept. Chem., 1:29:1 ('48). Growth of algae in water discussed, with particular reference to resulting changes in carbonate equil. observed in Brisbane water supply. Taste- and odor-producing organisms listed, together with photosynthetic products of different classes. Myxophyceae and Diatomales are principal offenders. To combat reduced filter runs, $CuSO_4$, Cl and Cl-$(NH_4)_2SO_4$ employed, latter preventing formation of objectionable-tasting chloroderivs. of essential oils. *Pleurococcus*, particularly, grows excessively under widely varying seasonal conditions, rapidly choking slow sand filters. Usual treatment is to apply 0.2 ppm. $CuSO_4$ in sedimentation basins or to individual filters, or to spray hypochlorite soln. on water (avg. dosage 0.5 ppm. Cl) when organisms principally floating on surface. *Spirogyra, Cladophora, Zygnema* and *Mougeotia* also grow vigorously in filters, their presence, however, being generally beneficial inasmuch as they release O, consume CO_2 and cause deposition of $CaCO_3$. Deposits in filters and in exptl. glass tanks contg. actively growing algae were found to consist principally of $CaCO_3$ with lesser amts. of $Mg(OH)_2$ and small percentages of $MgCO_3$, together with SiO_2 from identifiable diatom skeletons and Al_2O_3 from coagulant floc. Coagulation with alum practiced only during rainy season when water turbid and not during long periods of clear water and vigorous algal growth. Analyses of filter sand prior to rainy season showed $CaCO_3$ and $Mg(OH)_2$ contents of 0.09–0.20% and 0.04–0.08%, resp., it being estd. that 24,000 cu.yd. contained 30 tons of $CaCO_3$ and 12 tons of $Mg(OH)_2$. While applied water was unsatd. to $CaCO_3$, supersatn. occurred in 6' of water overlying sand, being greatest in proximity to floating masses of algae (*Spirogyra*) and heavy mat of algae on sand surface. Effluent at times is somewhat undersatd. owing to pptn. on contact with previously deposited $CaCO_3$ and to bacterial and algal respiration of CO_2 within sand bed. At other times, filtered water is satd. to $CaCO_3$, titratable base being 3–10 ppm. (calcd. as $CaCO_3$) less than that of applied water. During periods of sparse algal population and when alum is being applied, effluent frequently contains higher (up to 5 ppm.) concns. of titratable base than applied water, with appreciably lower pH values; this indicates that CO_2 production is sufficient to redissolve previously pptd. material. Net effect is to reduce amt. of Na_2CO_3 and $Ca(OH)_2$ required for corrosion control. None required during periods of high algal activity. Expts. on movement toward equil. when supersatd. and unsatd. waters shaken with powd. $CaCO_3$ confirmed above observations. Supersatn. due to algal activity (*Spirogyra* and *Cladophora*) was largely corrected in 15 min., equil. being substantially reached in 120 min. Although conditions were not favorable for pptn. of $MgCO_3$ or $Mg(OH)_2$, small but marked removal of Mg oc-

(*Continued on page 42*)

(*Continued from page 40*)

curred, presumably as double carbonate with $CaCO_3$. In unsatd. water, 120-min. contact with $CaCO_3$ insufficient to establish equil. Na aluminate is employed in conjunction with alum for 2 purposes: [1] to reduce chem. costs of treatment of water of low turbidity and high color; and [2] to maintain pH of applied water above isoelec. point (5.5) when high coagulant dosages required for turbidities up to 20,000 ppm. Latter is important to prevent pptn. of $Al(OH)_3$ in $CaCO_3$-contg. sand beds and consequent rapid increase in loss of head which cannot be corrected by scraping. When this occurs, subsequent filtration of waters with pH values of 7.4–8.0 gradually removes Al as aluminate. Operating data substantiate these conclusions. Deposits found on leaves of aquatic plants *Hydrilla verticillata, Potomageton perfoliatus* and *Vallisneria spiralis* in Brisbane R., probably formed when water was satd. or nearly so to $CaCO_3$, principally through activity of filamentous algae, were composed largely of $CaCO_3$, with some $Mg(OH)_2$ and $MgCO_3$. To det. general effects on water of active growth and decline of algae, samples were collected daily at 7 A.M. and 6 P.M. from glass tanks contg. *Spirogyra*. Results of analyses shown tabularly and graphically. pH increased during hours of photosynthesis and decreased overnight owing to bacterial and algal respiration of CO_2, which, although masked by effects of photosynthesis, occurs continuously. Titratable base decreased during day and increased during night owing to deposition and re-soln. of pptd. base, resp. Dissolved O increased during daylight hours to as high as 232% satn. and usually dropped to much below satn. overnight. Mg was definitely removed from soln., theoretically as double carbonate. In all analyses given, components were calcd. from equil. equations, math. derivation of which is discussed in some detail.

(*Continued on page 44*)

(Continued from page 42)

Summary of analyses of algae-induced deposits show that $Mg(OH)_2$ is constituent of those due to rapidly growing green algae but is absent from those arising from blue-green Myxophyceae. Frequent observations on 20-mi. stretch of Brisbane R. above impounding weir have shown progressive downstream increase in pH value from 7.5–7.6 to 7.9–8.1 and decrease in color (about 50%) and bacteria during low-flow period, despite abundant falling leaves. Lab. expts. indicate that this is due to coagulation of color in presence of Ca and Mg at higher pH values caused by algal activity, coagulum, which is redispersed at lower pH values, carrying down with it bacteria and plankton. *Caloglossa* (Rhodophyceae), never before reported in Queensland, has been found recently attached to rocks in shallow, slowly flowing tributary of Brisbane R. which receives very little direct sunlight. Lab. cultures displayed relatively weak photosynthetic activity.—*C.A.*

Algae Control and Methods of Enumeration. SAMUEL O. SWARTZ. J.N.E.W.W.A., **64**:64 (Mar. '50). Water works operators should have knowledge of algae and of methods of control. Algae may cause taste and odors, color, clogging of screens and filters, high chlorine demand and high pH. It is necessary to become familiar with various microscopic forms. "Algae chart" has been compiled by author and is used in bacteriological laboratory of Metropolitan Water Works, Boston, Mass. Used in conjunction with chart on which is recorded estimates of algae from microscopic examination. In making estimate, 255 ml. of water is passed through filter of white sand supported on 200-mesh silk. Sand is graded to pass 60-mesh screen and to be retained on 120-mesh. When sample is thus filtered, all algae caught in sand, which is then washed with 5 ml. of distd. water. Sand is allowed to settle and water is decanted to another tube and then examined microscopically. Taste and odor control can be realized by establishing limits for various undesirable organisms. When numbers approach limit, corrective measures may be inaugurated by chemical treatment.—*P.H.E.A.*

POLLUTION AND POLLUTION CONTROL

Report of the Sudbury Valley Commission Relative to the Sudbury River and Its Environs. Mass. House Doc. 2351 (Mar. 10, '50). This is report of four officials on sanitary conditions in Sudbury R. Valley, with total land area of 257,327 acres (402 sq.mi.) containing 2 cities and 34 towns, as well as water areas and extensive meadows and swamps. Proposals for action are reduction of floods, control of low water flow by reservoirs, removal of pollution, reduction of mosquitoes and restoration of recreational use. Population in area estimated at 171,594 as of Jan. 1, '50. Within 15 mi. of river is found population of 1,662,755. River controlled by number of dams, as well as by abandoned reservoirs formerly used by Boston for source of water. Weed growths, principally duck weed and water chestnuts, prevalent at low water flow, stimulated by nitrates from sewage filtration plant effluents. Pollution is substantially from sewage treatment works effluents of Framingham and Natick, and industrial waste of carpet mill at Saxonville. Marked pollution has occurred in Sudbury R., killing fish. If both towns and industry discharge to south metropolitan sewer, Sudbury R. will be suitable for recreation, including bathing. Assabet R. more industrialized and affects Concord R. Vigorous pollution control required. Mosquitoes great nuisance in valley but can be controlled

(Continued on page 46)

(Continued from page 44)

by ditching and spraying. River flow used for generating power for processing goods, also for irrigation of crops. Ponding has benefited fishing. Control of land use by zoning now mostly local but should be extended on broader basis. Fire control to prevent bog fires in dry seasons also needed. Along with dominant use for residence, provision for recreation most important, to promote bathing, fishing, boating and hunting, as well as stimulate wildlife. Adequate and uniform regulation of water area of river and its banks required. Farming most important commercial use. Some humus beds also available. Report contains 20 supplemental statements: [1] historical; [2] legislative; [3] hydraulic data; [4] sanitary conditions of rivers; [5] river problems; [6] Lemna Minor as aggressive weed; [7] mosquito control; etc. Under sanitary conditions of rivers, water supplies in area and disposal of sewage on Sudbury R. at 6 towns and 1 city, all served by separate system of sewers, discussed. Industrial wastes entering rivers generally from wool scouring and finishing woolen cloth. State of rivers in certain localities unsatisfactory under extreme conditions. Sudbury R. at times covered with heavy growths of duck weed, which decays and creates odor nuisance. Water unsuitable for certain industries at such time. Content of *Esch. coli* too high for safe bathing. Assabet and Concord Rivers unsafe for bathing. Concord R. unsuitable for industry. Population equivalent tributary to sewage works on entire watershed is 77,350. Effluent equivalent to 6,375 persons. If Framingham and Natick sewage effluent diverted, effluent equivalent population would be reduced to 3,035. Low flow in Sudbury R. is 1.5 mgd. to which is added 2.567 mgd. of sewage effluent. Conclusion is that minimum flow of 4.1

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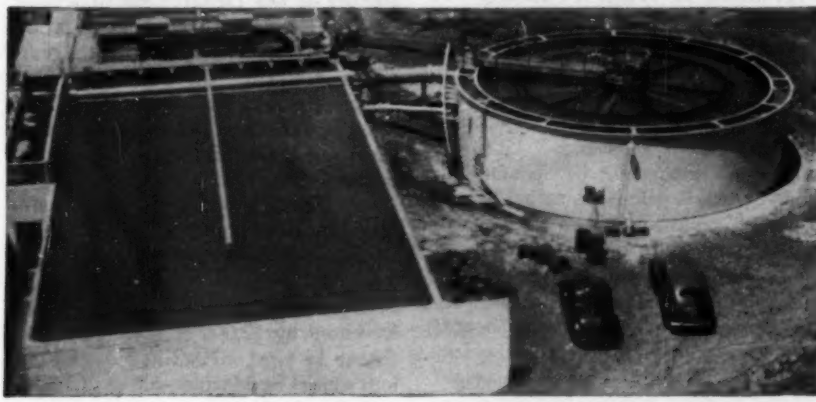

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(*Continued from page 46*)

mgd. should be maintained in Sudbury R. below Saxonville by utilizing available storage, after diversion of Natick and Framingham sewage. As to prevalence of duck weed in Sudbury and Charles Rivers, conclusion reached that alkaline sewage wastes have important bearing on weedy behavior of Lemna Minor in both rivers. —*P.H.E.A.*

Prevention of Water Pollution. V. W. Bacon. California's Health, **7**:185 (June 30, '50). Pollution control legislation enacted by '49 regular session of California legislature establishes newly defined concepts of "contamination," "pollution," and "nuisance." Responsibility for abatement of "contamination," which is defined as actual hazard to public health, vested in local health departments and state department of public health. Control of "pollution" and "nuisance" is responsibility of regional water pollution control boards created for 9 major drainage regions of state. "Pollution" and "nuisance" defined to mean adverse and unreasonable effects of disposal of sewage or industrial wastes which are not actual hazards to health. Thus, they represent economic aspects and effects of waste disposal. State water pollution control board established to administer operating, research and loan funds; to formulate statewide policy for control of water pollution with due regard to authority of regional boards; and to act as appeal board in any specific instance of pollution where it is found that regional board has not taken proper action.—*P.H.E.A.*

Health Aspects of Pollution Control Problems and Progress. Warren J. Scott. Conn. Health Bul., **63**:34 (Feb. '49). Brief account of factors affecting stream pollution in Conn. presented. Health aspects of potable water supply, bathing waters and shellfish-growing waters outlined. Problems of communities and industries considered. Financing, planning and construction difficulties mentioned. Conn. state water commission and two regional interstate commissions active. Public interest and opinion have grown. Statewide programs recommended to abate pollution on grounds of cleanliness of environment as well as public health.—*P.H.E.A.*

Report of the Department of Public Health Relative to an Investigation and Study . . . of the Causes Creating a Nuisance on the Shore and Beaches of Marblehead, Salem, Danvers, Beverly and Manchester and of Means of Rectifying Such Conditions. Mass. House Doc. 2236 (Dec. '49). This report studies area with shoreline of about 44.5 mi. in vicinity of Salem, Danvers, Beverly and Manchester. Various municipalities have resident population of about 100,000, increased by 12,000 summer residents. About 14 mgd. of sewage discharged into waters by Marblehead, Manchester and South Essex sewerage district. Latter contributes about 90% of total. Float tests indicated movement of sleek areas. Over 260 private sewer outlets noted. Beaches inspected after close of bathing season. Decaying organic matter found, including garbage. Bacterial tests indicated beaches safe at time examined. Further field investigation and enforcement of rules and regulations of department, prohibiting pollution of inland and tidal waters recommended.—*P.H.E.A.*

Preventing Stream Pollution. T. F. Wisniewski. Food Packer, **30**:62 (Jan. 49); Food Tech., **3**:6 (May '49). Wisconsin cracking down on canners. Committee on Water Pollution employs field men for experimentation. Continuous-flow, fill-and-draw chemical precipitation processes were developed. Biological, sand and carbon fil-

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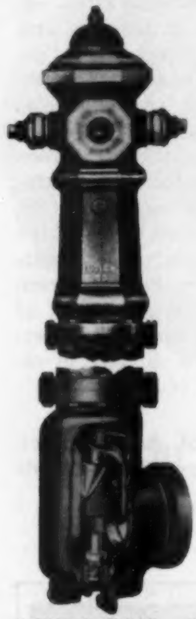

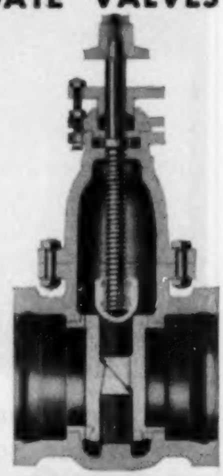

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tration, and effects of wastes on municipal systems have been investigated. Field men help canners select method most suitable for their type of operation. Lagooning and irrigation are most common where possible.—*P.H.E.A.*

Effect of Waste Disposal of the Pebble Phosphate Rock Industry in Florida on Condition of Receiving Streams. RANDOLPH C. SPECHT. Trans. Am. Inst. Mining Met. Engrs., **187**, Tech. Pub. 2878-H; Mining Eng., **187**:779 ('50). During '47 waste disposal totaled more than 6,000,000 tons of quartz sand and clay, deposited in large settling areas provided by individual operators. Clear effluents from operations were not found to be toxic and fish lived for 30 days in waters having turbidity of 80–130 ppm.—*C.A.*

INDUSTRIAL WASTE TREATMENT

Neutralization of Acid Wastes. B. W. DICKERSON & R. M. BROOKS. Ind. Eng. Chem., **42**:599 (Apr. '50). Solution to difficult waste acid neutralization problem outlined. Acids handled were nitric and sulfuric, and they varied widely in volume, concentration and ratio. Neutralization accomplished effectively in multiple-unit reaction chamber provided with 2-point pH-controlled addition of dolomite lime slurry. Use of multicompartment chamber eliminated effect of rate of flow. pH controllers employed immersion electrodes placed directly in reaction chambers.—*P.H.E.A.*

How to Dispose of Acid Wastes. J. E. COOPER. Chem. Indus., **66**:684

(*Continued on page 52*)

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(May '50). Acid wastes of pH below 4.5 corrode sewer lines, interfere with sewage treatment, kill fish. Cheapest treatment is dilution, if there is enough dilution water of sufficient alky. and it does not contain other pollutants like iron, copper, chromium. Sometimes waste and diluting water must be mixed artificially to prevent local acid concentrations. Neutralization of acid wastes is generally done with limestone beds. Upflow beds use 10–16-mesh stones, downflow 1–3" stones. High-calcium limestone acts more rapidly than dolomitic. Upflow beds, 18–24" deep, are operated at 40 gpm./sq.ft.; downflow beds, 5' deep, at 0.5–1 gpm./sq.ft. Water should be low in turbidity, free from fat and grease and have less than 0.5% H_2SO_4. Effluent has pH around 4.2 due to free CO_2; aeration can raise pH to 6 and higher.—*P.H.E.A.*

Experiences With Acid Neutralization of Waste Waters. BERNHARD KRATZ. Vom Wasser, **17**:83 ('49). Discussion of waste disposal problems which arose in conjunction with sudden production increase of trinitrotoluene plants during war years. Neutralization of large vols. of acid waste waters with lime, clarification, dewatering and calcination of mud, as well as various recovery methods, described.—*C.A.*

Treatment of Cotton-Finishing Waste Liquors. G. G. BOGREN. Ind. Eng. Chem., **42**:619 (Apr. '50). Pilot-plant experiments described were conducted as step in program to abate pollution of small stream receiving wastes from large cotton-finishing plant. Finishing plant has dual sewerage system—light wastes are discharged to stream without treatment, and heavier wastes are pumped to treatment plant. Wastes are now receiving primary and partial secondary treatment. Complete secondary treatment is necessary in planned stream pollution abatement program. Wastes vary widely in alky. from hour to hour. For this reason, and for economic reasons, high-rate trickling filters seemed most promising method of treatment. Pilot-plant experiments showed that B.O.D. removal of at least 60% was possible at rate of filtration of 10,000,000 gpd./acre.—*P.H.E.A.*

Treatment of Cotton Printing and Finishing Wastes. S. E. COBURN. Ind. Eng. Chem., **42**:621 (Apr. '50). Pilot plant was operated during '48 to find most feasible method of treating cotton printing and finishing wastes before discharge into Delaware R. Found that construction costs of high-rate trickling filter plant would be approximately $45,000 more than chemical treatment plant, but net annual charges would be about $25,000 less; difference in charges due largely to costs of labor and chemicals. High-rate trickling filters present fewer complications in operation and less serious sludge disposal problem. Results are given of analyses of wastes and treated effluent under varying operating conditions, and estimated costs of constructing and operating plant to handle 2,000,000 gpd.—*P.H.E.A.*

Treatment of Cyanide Wastes From the Electroplating Industry. ARTHUR N. CORCORAN. Sew. & Ind. Wastes, **22**:228 ('50). Concns. of CN^- exceeding 0.1 ppm. harmful to aquatic life whereas permissible concn. for sewage plant operation is 1–3 ppm. Ponding has limited application and is recommended only as temporary measure. Acidification with H_2SO_4 to pH of 4.0 and aeration can reduce wastes to 1.0 ppm. CN. Alkali treatment to pH of 8.5 and chlorination with 2.73 parts of Cl_2 to 1 ppm. CN

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will convert CN to CNO^-. Further chlorination will convert CNO^- to CO_2 and N_2. Electrolytic oxidation of strong wastes has created considerable interest because no sludge is formed and metals are recovered, but reaction very slow. Thiocyanate conversion has many drawbacks. Treatment with $FeSO_4$ and lime not popular in U.S. Ion-exchange methods not promising. —*C.A.*

Radioactive Waste Disposal. JOHN A. AYRES. AEC Doc. 2802 (Jan. 31, '50); Nuclear Science Abs., **4**: 345 (Apr. 30, '50). In labs. using radioactive tracers, large amount of liquid wastes having low level of activity will be produced. Often it may be desirable to treat these wastes before they are discharged into sewers or released in any other manner into ground or water. Ion exchange is applicable to problems of this sort in that it may be used for removal of small amounts of ions from very dilute solutions. Problem of removal of radioactive solids from lab. wastes is complicated by fact that wastes are heterogeneous and vary from day to day. Wastes will contain solids, organic solvents, oils and reagents which may form complexes with metallic ions. Research program to evaluate ion exchange has been divided into several parts: [1] efficiency of ion-exchange resins for this type of process; [2] determination of operating curves under set conditions for typical types of ions; [3] determination of amount of leakage or efficiency of ion-exchange resin in order to estimate decontamination factors; [4] effect of reagents which might cause precipitation or complexing; [5] effect of solvents, greases, detergents or precipitates; [6] possible concentration by incineration. 2 procedures, complete and partial deionization, are described. Experimental evidence was obtained to show: [1] high decontami-

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nation factor may be obtained using synthetic solutions which represent laboratory conditions; [2] high concentration factor may be obtained making possible compacting of wastes into small volumes; [3] presence of ordinary contaminants such as oils, solvents and precipitates does not have greatly deleterious effect.—*P.H.E.A.*

Separation of Oil Refinery Waste Waters. R. F. Weston. Ind. Eng. Chem., **42**:607 (Apr. '50). Problem of oil removal from oil refinery waste waters discussed. Problem includes separation, collection and reconditioning of oil for recharging to refinery processes, and also treatment of waste water to make it satisfactory for disposal. Oil is separated from waste waters using gravity differential-type separators. Efficiency of this operation is based on relative contents of that portion of oil amenable to separation by gravity differential flotation processes. Analytical procedure to determine "susceptibility to separation" was devised to find nonseparable oil content of sample. Efficiencies of various separators used are presented. After oil is separated it will contain suspended solids and water. These materials may interfere with efficient processing and it is necessary to treat slop oils prior to recharging to processing equipment. Typical data on characteristics of slop oil treatment given. Operating data given for automatic backwash sand filter installation that is successfully "polishing" separator effluent. Pilot-plant studies using biological filters indicate general improvement of separator effluents, including substantial reductions in oil content.—*P.H.E.A.*

Underground Disposal of Process Waste Water. L. K. Cecil. Ind. Eng. Chem., **42**:594 (Apr. '50). Petroleum industry has solved problem of disposal of oil field brines by injection into underground formations. Chemical industries with wastes that cannot be treated for disposal in nearby diluting waters may find this method applicable. General discussion of factors involved in finding formation suitable for injection wells and limitations of wastes quality for such disposal may be of assistance in deciding if this method should be investigated by individual chemical plants. Report of 18 months' operation of waste treatment plant and disposal well points out some problems to be expected in preparing new type of waste for injection well disposal.—*P.H.E.A.*

Treatment of Compressed Yeast Wastes. W. Rudolfs & E. H. Trubnick. Ind. Eng. Chem., **42**:612 (Apr. '50). Compressed yeast wastes have been treated in compact plant comprising anaerobic digestion and trickling filters, with resultant B.O.D. reductions of 80–98% by means of careful control of loading and other factors, including, for digesters, acclimatization of seed sludge, maintenance of proper proportions of seed and substrate, and provisions for adequate contact between seed and substrate; and, for trickling filters, maintenance of proper concentration and neutral pH value in filter influent. Any desired degree of treatment can be attained in either unit by control of applied loading, so that loading is useful yardstick in evaluating plant performance, and treatment plant can be designed on basis of B.O.D. reduction requirements. Maximum loadings attained with either unit compare with results obtained in sewage treatment under similar conditions. —*P.H.E.A.*

Rapid Putrefaction in the Treatment of Highly Polluted Plant Wastes. H. Jung. Vom Wasser, **17**:38 ('49). Old method of artificial

(*Continued on page 58*)

(*Continued from page 56*)

biol. waste treatment has found some new applications in purification of some waters with high concns. of suspended or dissolved org. matter. Treatment of waste water from strawboard plant presents good example; plant effluent contains $Ca(OH)_2$ as well as large quantities of straw in finely dispersed and colloidal form. This water has pH of 10 or higher and $KMnO_4$ demand of about 5000 ppm., i.e., about 15 times that of ordinary domestic sewage. Accelerated biol. decompn. treatment was developed consisting in recirculation of large portions (up to 50%) of decompd. mud at controlled temp. of about 30°. Investigations were carried out on lab. scale and then in continuous operation on plant scale. Limitations of method, as well as performance data and economical factors, discussed.—*C.A.*

New Pumping and Piping System for Abrasive Industrial Waste Eliminates Pollution of River. P. D. OESTERLE. Heating, Piping & Air Cond., **22**:100 (July '50). Describes how industrial plant has met problem of avoiding stream pollution by new pumping and piping installation for disposing of highly abrasive waste resulting from grinding and polishing of plate glass. Waste, formerly allowed to flow into Allegheny R., now goes to thickener unit, from which clear water flows to river. Slurry is pumped through mile-long pipeline over 320' hill to settling basin formed by damming natural hollow.—*P.H.E.A.*

BOILERS AND FEEDWATER

A System of Water Treatment for Boilers up to 900 psi. With Low Makeup. I. Preparation of Raw-Water Makeup and Evaporation Operation. CHARLES L. WOLFF & IRVING LEIBSON. Southern Power & Ind., **48**:8:64 ('50). Most raw waters require pretreatment before evapn. Hardness, high alky., etc., are best removed by means of sludge blanket softener. Where carbonate concn. is higher than that equiv. to Ca, $CaSO_4$ can be added to ppt. $CaCO_3$. Where H_2S is present as sulfide, water can be chlorinated and excess Cl removed with sulfite or thiosulfate. Last traces of volatile materials can be removed in tray-type deaerating evaporator; if NH_3 present, condensate from vent condenser should be discharged to sewer. Evaporator operation discussed; continuous blowdown necessary, and cond. recorders should be used on both condensate and blowdown line.—*C.A.*

The Treatment of Water With Phosphate to Stabilize the Carbonate Hardness in Systems With Recirculation. I. N. OZHIGANOV. Izvest. VTI (Vsesoyuz. Teplotekh. Inst.), **16**:6:15 ('47); Chem. Zentr. (Russian Zone Ed.), **I**:1147 ('48). Addn. of *o*-phosphate in various forms to circulating water prevents formation of boiler scale in heat exchangers by raising crit. $CaCO_3$ concn. Addn. of 2–3 ppm. of P_2O_5 prevents pptn. of $CaCO_3$ at 60°. By such treatment, permissible carbonate hardness in circulating water can be increased to 280 ppm. CaO.—*C.A.*

Sodium Tripolyphosphate in the Quality Improvement of Water. GERHARD AMMER. Vom Wasser, **17**: 128 ('49). In contrast to *o*-phosphates, addn. of cold dil. solns. of polyphosphates to hard waters does not cause pptn., but causes stabilization of hardness compds. by inhibition of formation of crystal nuclei. Of polyphosphate series, $Na_5P_3O_{10}$ is not yet generally acceptable in water industry although it has several favorable properties. It is not deliquescent nor hygroscopic and thus can be shipped and

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proportioned without difficulty. pH of 1% soln. is between 8.6 and 9, i.e., corrosion effects on iron proportioning equipment are insignificant. Efficacy of compd. studied by comparison with more generally use $Na_6P_6O_{18}$. Under exptl. conditions used, stabilizing effect of both polyphosphates practically equal, although Bell reported more favorable results with $Na_5P_3O_{10}$ under somewhat different conditions—*C.A.*

Hideout of Sodium Phosphate in High-Pressure Boilers. F. G. STRAUB. Trans. A.S.M.E., **72**:479 ('50). High-pressure boilers operating at pressure of 1600–1700 psi. tend to suffer loss of phosphate from soln., which "hides out" or deposits in insol. form on tubes. This is due to formation of insol. form of Na phosphate, stable at temps. above 620°F., which is scale forming and resistant to heat transfer. Different nature of this phosphate from that usually present in boiler water has been confirmed by chem. means, x-rays and petrographic examn. Chem. analyses indicate presence of relatively high percentage of Na *o*-phosphate with Na hexametaphosphate, Na pyrophosphate, and Na trimetaphosphate each amounting to about $\frac{1}{8}$ as much as *o*-phosphate. Tests indicate that no insol. K phosphate is formed and that if suitable K-Na ratio is maintained, this insol. salt will not form.—*C.A.*

Chemical Treatment, Demineralization or Evaporation for Makeup in High-Pressure By-Product Steam Plants. J. D. YODER, W. L. WEBB & T. BAUMEISTER. Trans. A.S.M.E., **72**:491 ('50). Increased power production obtained by exhausting high-pressure turbines directly to process rather than to coils of evaporators, which in turn produce process steam, is demonstrated. Typical heat balances are presented for systems employing evapd. makeup in closed cycle and for 100% makeup cycles with water treatments consisting of [1] silica removal and Na zeolite softening, and [2] demineralization and silica absorption. Performance data and costs are given for both evapd. makeup and 100% treated makeup 1400-psi. cycles at Deepwater Operating Co.'s station.—*C.A.*

Sulfite and Silica in Boiler Water at Springdale. L. E. HANKISON & M. D. BAKER. Trans. A.S.M.E., **72**:505 ('50). H_2S was first detected in gases discharged from air ejector of main condenser of unit at Springdale Sta. about 6 mo. after unit placed in service. Investigation indicated that this was probably due to decompn. of Na_2SO_3, which appeared to be present in excessive amts., depositing on boiler tubes, where it decompd. at red heat to give sulfide, which in turn reacted with water to give H_2S. Reduction of Na_2SO_3 concn. did not eliminate H_2S production, so acid cleaning was resorted to for removal of deposits, after which boiler was brought to satisfactory operating condition. Severe corrosion due to H_2S was noted at discharge throat to condenser, and some corrosion of tubes in air ejector condenser. Erratic results for Na_2SO_3 analyses were found to be due to use of Cu sampling lines, which reduced Na_2SO_3. Monel was also reactive, so all sampling lines and condensers should be of steel. Sulfite detn. may be in error if samples are collected at or above 80°F. Silica contamination of feedwater system and turbine deposits may occur owing to fly ash that enters open vents.—*C.A.*

Feedwater Treatment During Early Operation of Steam-Electric Stations. R. C. ALEXANDER & J. K. RUMMEL. Trans. A.S.M.E., **72**:529 ('50). Based on experience with 5

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high-pressure steam-generating units at harbor steam plant, basic practices have been developed to follow during early operation of such installations. All available surfaces of feedwater system are cleaned by hand, and entire system is then cleaned by recirculation of hot detergent. Acids and dissolved O in feedwater system are kept at min. by proper venting of condensers, etc. Boiler water pH is adjusted to highest practical value (8.5 to 9) and cond. of condensate maintained at as low value as possible. Carbonates are maintained at min. by close attention to maintaining low alky. in makeup water to evaporators and min. condenser leakage. Sampling schedule is outlined for checking dissolved-O removal, pH, cond., Fe and Cu, etc.—*C.A.*

The Quality of Steam Condensate as Related to Sodium Sulfite in the Boiler Water. R. C. ALEXANDER & J. K. RUMMEL. Trans. A.S.M.E., 72:519 ('50). Data obtained from boilers operating at 900 psi. and higher indicate that it is probable that when more than 5–8 ppm. of Na_2SO_3 is present in boiler water, quality of steam will be affected. This steam compn. change will increase with increasing Na_2SO_3 concn. and will also be affected by alky. and pH of boiler water. pH is lowered, cond. increased, and reducing material in steam condensate, resembling H_2SO_3, is increased. When steam contains sufficient NH_3 or other alk. material to overbalance acid effects of CO_2, it is probable that, with small amts. of Na_2SO_3 in boiler water, point of lowest cond. of steam condensate will be near pH of 8. Near this point, increasing Na_2SO_3 concn. will lower pH and raise cond. of condensate, but lowering Na_2SO_3 concn. will increase both pH and cond. Test data are of special interest when cond. readings are used to est. solid-matter content of steam condensate. Similar

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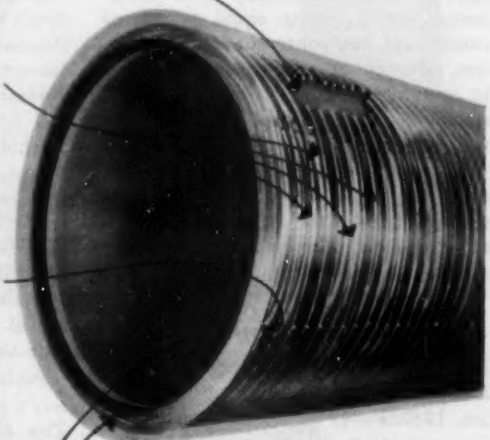

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studies should be made with boilers operating at pressures below 900 psi.—*C.A.*

An Automatic Degasser for Steam Sampling in Power Plants. H. M. RIVERS, W. H. TRAUTMAN & G. W. GIBBLE. Trans. A.S.M.E., **72**:511 ('50). Elec. cond. procedures are widely preferred for measuring purity of condensed steam, but gases such as CO_2 and NH_3 in condensate make interpretation of results somewhat uncertain. New degasser has been developed which automatically splits steam sample into 2 approx. equal streams of condensate, one contg. all solid impurities, other contg. all gaseous impurities in original sample. Heat-exchange elements are combined in unique way which insures that wt. ratio of one stream to other will remain substantially const. Both streams pass through cond. cells so degree of contamination can be measured continuously.—*C.A.*

TREATMENT—GENERAL

Decarbonation Experiments in the Laboratory. J. LEICK. Vom Wasser, **17**:123 ('49). Discussion of preliminary expts. for treatment of water which are considered desirable prior to large-scale operations. Certain exptl. conditions must be maintained to obtain results reproducible in actual operation. Details given on calcn. of residual carbonate hardness, form and quantity of lime addn., and exptl. arrangement.—*C.A.*

Decarbonation of Water in Quick-Reactors. J. LEICK. Vom Wasser, **17**:114 ('49). Quick-reactors for decarbonation of hard waters are preferred to clarification and settling tanks because of low space requirements and because of simplified sepn. of granular reaction product. Reactors are cone shaped and contain charge (fine sand or marble) grain size of which should not be less than 0.3 mm. and not in excess of 2 mm. Min. and max. capacity of reactors is limited by grain size and height of charge. With min. grain size of 0.3 mm., no carryover takes place at upward flow velocity of 20 m./hr. Charge is kept in const. motion by flow velocity; spent charge can be removed at either top or bottom. Turbulent flow within reactor is independent of method of injection of water, i.e., tangential or central. It is important only that injection stream is broken up by direct contact with charge and that no channeling takes place owing to pocket formation. Capacity of reactors under normal operating conditions is about 100 l./hr. per cu.m. of reactor vol. Turbidity of effluent should be as low as possible and is dependent on grain size and height of charge, quantity of pptd. $CaCO_3$, interference with deposition of $CaCO_3$ from dissolved org. impurities, temp. (optimum 5°–30°) and alky. Clearest effluents were obtained with lime water (rather than lime slurry) at as low pH as possible. —*C.A.*

The Removal of Manganese From Potable and Industrial Waters. H. BORNER. Vom Wasser, **17**:103 ('49). Because of stability of bivalent Mn salts in acid or neutral solns. Mn must be removed by aeration of water which has been made distinctly alk. $MnO_2 \cdot nH_2O$ added to water tends to remove Mn compds. by adsorption. Extent of adsorption is governed by pH value, temp., ratio of surface areas and degree of hydration of MnO_2. Some bacteria and spores also tend to remove Mn from water by converting bivalent Mn to quadrivalent $MnO(OH)_2$ and depositing latter in cells. Bacteria grow preferably in neutral or weakly acid waters at 18°–25°. Mn removal by bacterial action requires absence or

(*Continued on page 66*)

(*Continued from page 64*)

low concns. of certain cations (Na, K, Mg) or anions (I, Br) which have toxic effect and inhibit growth of cultures. Process details are given on phys., chem. and biol. methods of Mn removal, and economic aspects are discussed.—*C.A.*

Removal of Silicon Compounds From Water. W. C. BAUMAN. U.S. 2,510,855 (June 6, '50). Method described for removal of dissolved Si compds. in amts. corresponding to SiO_2 4–6 ppm. and dissolved salts in amts. chemically equiv. to 10–14 grains of NaCl per gal. H_2O. Procedure comprises forming intimate mixt. of finely divided fluoride of Ca, Sr or Mg and granular cation exchanger in acid form. Water contg. dissolved Si compds. is passed through this bed and thence through bed of substantially water-insol. basic form of anion exchanger. Water is substantially free of all ionizable inorg. impurities. Mesh of bed constituents, rate of flow of water and method of regenerating bed are described in detail. Flow diagram illustrates various steps of process.—*C.A.*

OTHER ARTICLES NOTED

Recent articles of interest, appearing in American periodicals, are listed below.

Controlling Water Supply and Sewerage in Subdivisions. GEORGE L. HALL. Pub. Wks., **81**:10:40 (Oct. '50).

What You Should Know About Fluoridation of Water Supplies. HENRY F. MUNROE. Pub. Wks., **81**:9:39 (Sept. '50).

Viability of *Escherichia coli* in Sea Water. RALPH F. VACCARO ET AL. Am. J. Pub. Health, **40**:1257 (Oct. '50).

Tularemia in Man From a Domestic Rural Water Supply. W. L. JELLISON ET AL. Pub. Health Repts., **65**:1219 (Sept. 22, '50).

Rainmakers Need a Better Yardstick. Interview With Wallace E. Howell. Eng. News-Rec., **145**:13:34 (Sept. 28, '50).

"Getting the Evidence" on Evaporation. ANON. Reclamation Era, **36**:192 (Oct. '50).

Technical Skills for Soil and Water Conservation. H. H. BENNETT. Scientific Monthly, **71**:248 (Oct. '50).

Hydrographic Factors Involved in the Dispersion of Pollutants Introduced Into Tidal Waters. BOSTWICK H. KETCHUM. J. Boston Soc. Civ. Engrs., **37**:296 (July '50).

Persistence of Chlorophenols in Polluted River Water and Sewage Dilutions. M. B. ETTINGER & C. C. RUCHHOFT. Sew. & Ind. Wastes, **22**:1214 (Sept. '50).

The ABC's of Prestressed Concrete. DEAN PEABODY JR. J. Boston Soc. Civ. Engrs., **37**:315 (July '50).

Service Lines

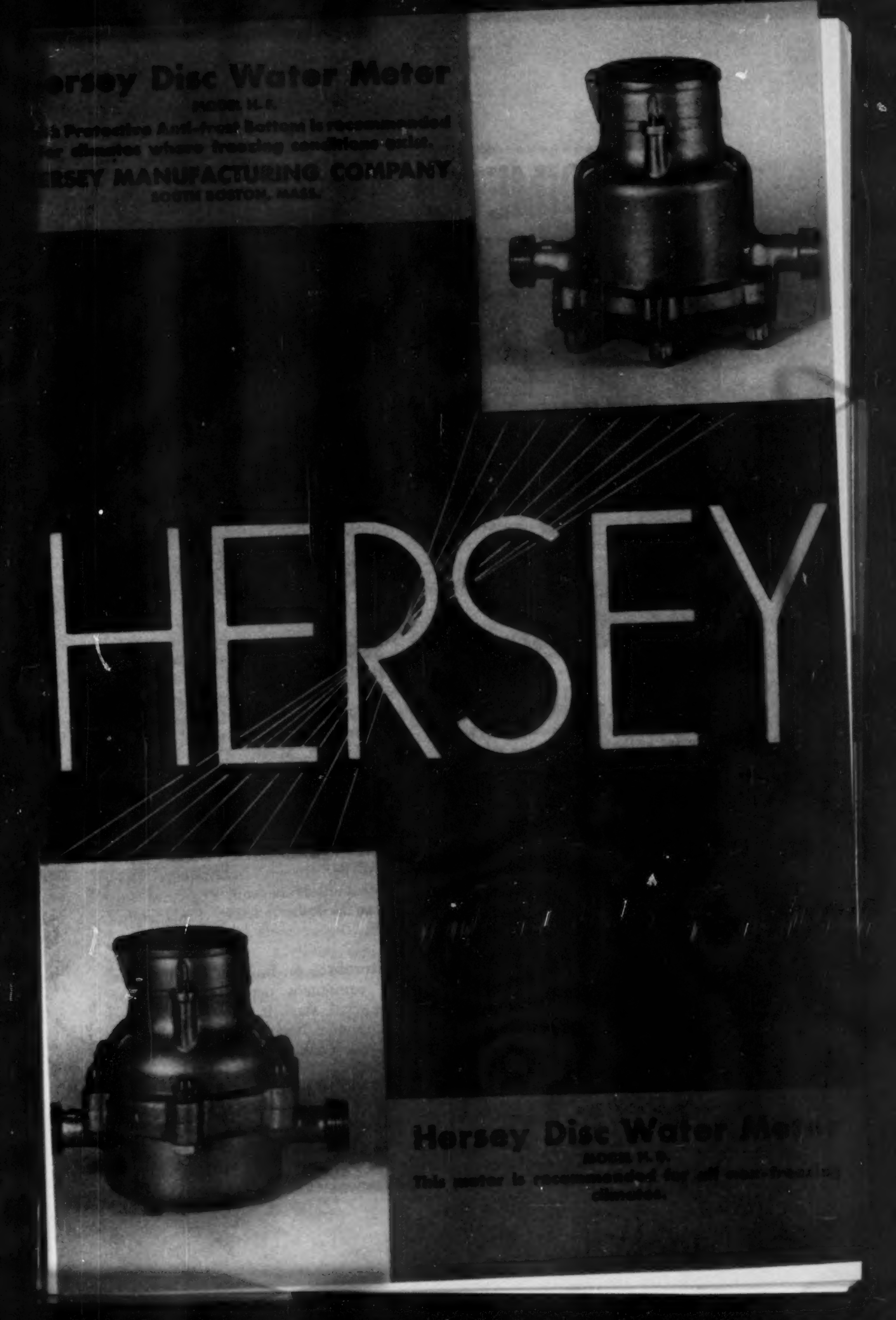
"**Corrosion** of Brass Pipe and Copper Tubing" and the use of sodium silicates to check such attack in water services is discussed in a leaflet distributed by Water Service Labs., Inc., 423 W. 126th St., New York 27, N.Y.

Treatment of boiler feedwater by the Hot-Z method of Worthington Pump & Machinery Corp., Harrison, N.J., is discussed in Bul. W-212B7, a six-page folder available upon request. Among topics covered are filtration, recirculation, backwashing and sludge removal.

"**Counters,**" a reprint of an article on the operation of Geiger counters by Serge A. Korff, may be obtained for 10¢ from V. W. Palen, New York University College of Eng., New York 53, N.Y. Special and industrial applications of the counters are mentioned.

A slide calculator for water flow in pipes 3 to 16 in. in size is available upon request from Grinnell Co., Inc., 270 W. Exchange St., Providence 1, R.I. The universal calculator was designed by T. Francis O'Connor and relates size and length of pipe, friction coefficient, flow, velocity and pressure loss in both pounds and feet. Directions for using the handy pocket-size calculator are included, as is also a reprint of a classic article by O'Connor from the 1943 *Proceedings* of the Maryland-Delaware Water & Sewerage Assn., entitled "A Rapid Method of Analyzing Flow in Water Distribution Systems Based on the Theory Developed by Professor Hardy Cross."

(*Continued on page 70*)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(Continued from page 68)

A pump sequence control system which can start or stop pumps as required by changes in flow demand is described in Bul. A-8A-4 available from Fischer & Porter Co., County Line Road, Hatboro 4, Pa. The device is actuated by an electronic circuit and is designed to handle flows of from 0.5 to 6,000 gpm.

A new source of microscope field illumination using an ordinary 40-w. light bulb is the Bausch & Lomb Micro-Lite. A circular, D-1020, may be obtained from the company at Rochester 2, N.Y.

"pH Electrodes," their assembly, replacement and use are the subject of a 28-page catalog, EN-S5, offered by Leeds & Northrup Co., 4934 Stenton Ave., Philadelphia 44, Pa. Accessories and supplies are also listed.

Sodium aluminate treatment for softening and clarification, as well as boiler feedwater treatment, is the subject of a new booklet issued by the Merrimac Div., Monsanto Chemical Co., Everett, Mass.

Pneumatic transmission for the recording of flow, pressure or temperature conditions is discussed in Instrumentation Data Sheet 9.1–4, issued by Brown Instruments Div., Minneapolis-Honeywell Regulator Co., Wayne & Roberts Ave., Philadelphia 44, Pa.

Literature on handling of fluorine with equipment made of Monel or Monel-clad steel is being offered by International Nickel Co., Inc., 67 Wall St., New York 5, N.Y.

Corrosion-proof cements are the subject of Bul. 5-1 of Atlas Mineral Products Co., 43 Walnut St., Mertztown, Pa. A chart aids in the selection of the proper material for various applications.

Section Meeting Reports

Rocky Mountain Section: The twenty-fourth annual meeting of the Rocky Mountain Section was held at the La Fonda Hotel in Santa Fe, N.M., on September 28–29. The total registration, including members and guests, was 126.

On Thursday morning, September 28, Lieutenant Governor Montoya welcomed the group. The chairman, B. V. Howe, responded and then appointed the various committees. Harry Potts, water rights engineer

Palefaces attempt Indian Buffalo Dance at Rocky Mountain pow-wow while amused expert tolerantly supplies rhythms.

for the Board of Water Commissioners, Denver, described the method of forecasting runoff in an excellent paper entitled, "A Photographic Snow Survey." This paper received considerable discussion.

In the afternoon A.W.W.A. President W. Victor Weir spoke on "The American Water Works Association and You." This was followed by presentation of papers by John Bliss, state engineer of New Mexico, and R. T. Littleton of the U.S. Geological Survey, Cheyenne, on "New Mexico Underground Water Laws and Administration" and "Adjudication of

(*Continued on page 76*)

(Continued from page 72)

Ground Water—Wyoming." "Fluoridation of Water Supplies" was described by Robert Downs, director of public health, dentistry division, for the Colorado State Board of Health. Considerable discussion followed. A movie film entitled "Prestressed Concrete" was shown by the courtesy of the Preload Central Corp., Kansas City, Mo., and closed the Thursday session.

On Friday, September 29, the morning session opened with a paper entitled "Ecology of Organisms in Surface Water" by C. M. Palmer and C. M. Tarzwell of the Environmental Health Center, U.S. Public Health Service, Cincinnati. Discussion was led by George J. Turre, sanitary engineer with the Denver Board of Water Commissioners. The "Methods of Automatic Control of Pumps" by G. E. Riepe, Builders-Providence, Wilmette, Ill., was then presented, followed by discussion of the paper. The remaining time of the morning was taken up by a talk presented by

Retiring chairman B. V. Howe welcomes successor Charles G. Caldwell to speaker's platform at Rocky Mountain Section banquet (left) as the H. L. McLaughlins and the John Burgesses look on (right).

Frederick F. Fish, biologist with the U.S. Public Health Service at Dallas, Tex., on the subject of "The Aquatic Life Balance in Reservoirs."

At noon on Friday, September 29, a business luncheon was held at the La Cocina. The Friday afternoon session consisted of papers on "Factors Influencing the Efficiency of Activated Carbon" by Al Hyndshaw, Industrial Chemical Sales Div., Tyrone, Pa.; "Water Works Accounting" by Elmer F. Carter, chief of operation, Public Service Commission, Public Utility Div., Santa Fe, N.M.; and "City Planning and Utility Coordination" by Allen M. Voorhees, city planning engineer, Colorado Springs, Colo. In addition, two films were shown, entitled, "36-in. Valve Insertion in a 36-in. Cast-Iron Main" by courtesy of A. P. Smith Mfg. Co., East Orange, N.J., and "Trenton Cleans Its Water Mains" by courtesy of the National Water Main Cleaning Co., New York.

The annual banquet was held Friday evening. James R. Scott, New Mexico director of public health, presided as toastmaster. Attendance at

(Continued on page 78)

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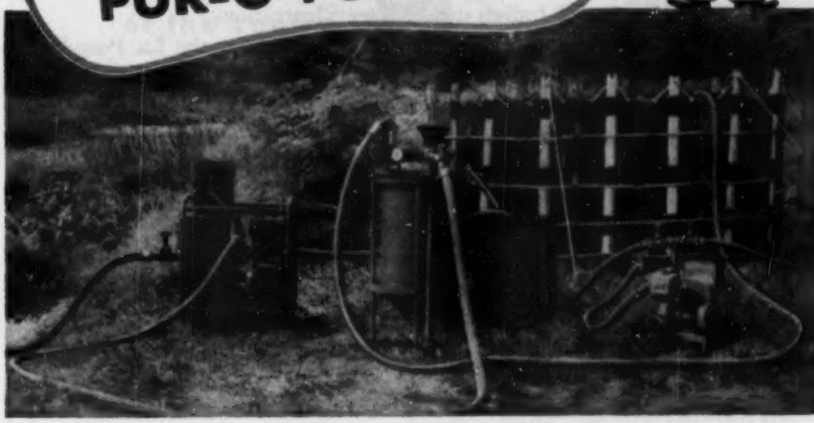

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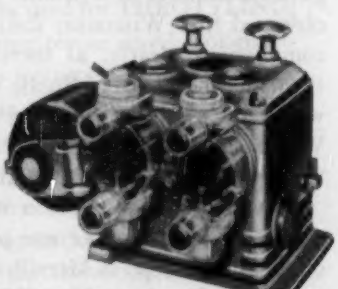

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(Continued from page 76)

the banquet numbered 146; music was furnished for the occasion by a colorful Spanish orchestra. After the banquet, entertainment was provided by a group of Indian dancers from Los Alamos, under the direction of C. S. Defandorf, who also participated in the San Juan Indian dances. During the periods while the dancers changed costumes, Dr. Defandorf selected certain individuals from the audience, who assisted him in presenting some of the colorful Indian dances (see illustrations).

On Thursday evening a cocktail party was held at the La Posada Inn by courtesy of Herkenhoff and Turney, consultants of Santa Fe, N.M. This was very much enjoyed by those who attended.

A luncheon was served for the ladies at the La Cocina. An interesting trip around old Santa Fe followed the luncheon; this trip was greatly enjoyed by the ladies.

During the evening Spanish dance music was rendered by the hotel orchestra. An excellent program of entertainment was presented and enjoyed by all who attended the meeting in Old Santa Fe—at the End of the Trail.

George J. Turre
Secretary-Treasurer

Wisconsin Section: The 29th annual meeting of the Wisconsin Section was held at the Hotel Raulf in Oshkosh on September 19–21, with a total attendance of 290.

The morning of Tuesday, September 19, was devoted to registration, and the opening session was held that afternoon, with Chairman Thomas M. McGuire presiding. The president of the city council appeared and gave the address of welcome. In his opening remarks, Chairman McGuire observed that Wisconsin statutes did not define the powers of the utility commissions clearly, as there is some overlapping of authority between them and the Board of Public Works. A joint committee has been selected, with representatives from the water and electric utilities and the League of Wisconsin Municipalities, to work out tentative legislation for proposal to the next session of the legislature, to remedy this difficulty.

A Distribution Session was then held, with Edward F. Tanghe presiding. The first paper was presented by Hubert J. Evers, superintendent of the Water Dept. at Merrill, entitled "Problems in Changing From a River to a Well Supply at Merrill." The next paper, "Distribution Problems," was presented by Walter A. Peirce, manager of the Water Dept. at Racine, who outlined the equipment which was necessary for maintenance of the distribution system and the valve spacing and hydrant location in Racine.

The members of the panel discussion on distribution system problems were the two speakers, together with Superintendents Arthur J. Jark, of Jefferson; Henry A. Ehlers, of Antigo; Everett Westphal, of Neenah; and

(Continued on page 80)

(*Continued from page 78*)

Harry Draves, of Michigan City, Ind. The matter of surface water in hydrants was discussed and it was recommended that hydrant drips be closed and the hydrant pumped out. One speaker found it advantageous to have auxiliary valves on all hydrants, and it was agreed that there should at least be auxiliary valves on all hydrants in the high-value district.

During the discussion of radio communication, various methods of sharing transmission facilities were described. At Madison the Water and Fire Depts. cooperate to share the same band, but at Sheboygan the Police Dept. refuses to allow water utility use of its system, although presumably such use would be permissible when public safety is involved. At Waukesha a two-way radio system includes a special water utility band. It was noted that the advantages of radio communication were quite clear in such an emergency situation as developed in Winnipeg during the flood this spring.

The necessity of reinforcing the distribution system in rapidly growing cities was emphasized, and it was pointed out that plans should be made for anticipated growth, so that when mains are installed, proper sizes will be used. There is a definite trend toward mechanical-joint pipe, with cast-iron bolts and nuts and rubber gaskets tipped with lead to provide the electrical continuity which is necessary for thawing out mains and services by electricity. A show of hands indicated that about 75 per cent of the municipalities are using mechanical-joint pipe.

The first paper in the Surface Water Session on Wednesday morning, "Proposed Fox River Valley Pipeline From Lake Michigan," was given by Bruno Hartman, president of the Jerry Donohue Eng. Co. of Sheboygan. Andrew J. Marx of Menasha discussed "The Pretreatment Basin for Algae Removal" and explained the type of construction adopted in Menasha which permitted the handling of Lake Winnebago water in a much more satisfactory way and at a lower cost than before.

A luncheon was held Wednesday noon at which A.W.W.A. President W. Victor Weir emphasized the necessity of replacing obsolete equipment, even though it was not worn out, when the increased efficiency resulting would more than absorb the additional fixed charges. He recommended that every superintendent check over his equipment from this standpoint and make the necessary changes. He suggested this as the best method of reducing costs, in view of the rapidly increasing cost of labor and materials.

Harold Londo, superintendent of the Water Dept. at Green Bay, presided at the Ground Water Session. The first paper, "Deep Well Turbine Pump Efficiencies," was presented by A. O. Fabrin, chief engineer of Layne & Bowler, Memphis. A "Progress Report on Ground Water Survey in Wisconsin" was next given by W. J. Drescher, hydraulic engineer of the U.S. Geological Survey at Madison, who outlined the work which is being

(*Continued on page 84*)

(Continued from page 80)

done as a joint project of the survey in cooperation with the University of Wisconsin. Work has been done on both shallow wells and on deep wells, particularly in the critical deep well areas found at Milwaukee, Wauwatosa and Waukesha, as well as at Green Bay.

The panel discussion for this session was led by Mr. Londo. The first subject discussed was "Electrologging of Wells," by G. M. Galloway of the Layne-Northwest Co., who showed an electrolog of a new well recently developed at Madison. He emphasized the effectiveness of this log in determining where the well should be shot. The second subject, "Maintenance of Shallow Wells," was discussed by V. W. Baker, superintendent at Wisconsin Rapids, who spoke on the various types of chemical treatment used to restore wells to capacity. The third subject, "Bazooka Shooting of Wells," was a contribution of G. L. Smith of the G. L. Smith Co. In a comparatively recent development, a large number of "bazooka" bombs attached to a round aluminum frame are lowered into the hole and then detonated. Advantages over the previous method of shooting with explosives include 15–20 ft. penetration into, rather than shattering of the rock formation; reduction of hazard; and lessening of required bailing to such a degree that it is not necessary to have a well rig on the job, as the amount of sand loosened by the shots is very small.

A lively discussion developed when O. J. Muegge, state sanitary engineer, stated that the Wisconsin statutes provide that permission must be obtained for any well development with a capacity in excess of 100,000 gpd. He interpreted the statutes to mean that the revamping or repair of any well to increase a capacity already in excess of 100,000 gpd. required the approval of the state sanitary engineer. Representatives of well drilling companies and companies selling deep well pumps objected that unless the legislation was liberally interpreted, industrial plants requiring water for their processes would be driven from the state. Muegge stated that it was his duty to enforce the statutes and that the only possible remedy would be the revision of the law by the legislature.

The banquet was held Wednesday evening, September 20. Dr. John A. Schindler of Monroe, who has received national publicity because of his talk on "How to Live to be 100 Happily," called attention to the fact that many of our ills were caused by worries, troubles and cares, but that they were just as real and painful as if they were organic diseases.

Chairman Zenno A. Gorder presided at the general session Thursday morning. The first paper, "Does Your Water Works Work for Nothing?" was presented by C. MacDonald, superintendent of the Meter Div. at Elgin, Ill., who recommended that accurate meters be installed on the discharge of all pumps and that the distribution system be completely metered. He stated that if this were done and the meters properly maintained, the

(Continued on page 86)

(Continued from page 84)

unaccounted-for water would be reduced to a minimum. He discussed in considerable detail the methods of testing and repairing meters in Elgin.

The second paper was given by Horace R. Frye, superintendent at Evanston, Ill., on the "Mechanical Phases of Fluoridation." He outlined the various types of plants in use in Wisconsin and northern Illinois and showed lantern slides to illustrate his talk. Discussion of each of the foregoing papers held in panel form, was by Harold T. Rudgal of Kenosha, Harold Londo of Green Bay and J. O. Fortin of Glen Ellyn, Ill., on the first paper; and Leon A. Smith of Madison, Jerome Zufelt of Sheboygan and J. J. McCarthy of Racine, on the second.

The Wisconsin and Illinois Sections cooperate very closely. The Illinois Section meets in the spring and the Wisconsin Section in the fall, and invitations and programs are mailed out before each meeting to the members of the adjoining sections. There is also an interchange of speakers, with at least two Illinois men on each Wisconsin Section program and at least two Wisconsin men on the Illinois Section program.

It has been the policy of the Wisconsin Section for several years to open each session with a movie, not necessarily technical in nature. By this means, the meeting room is well filled by the time the meeting is called to order and there is a minimum of confusion after the technical session starts.

LEON A. SMITH
Secretary-Treasurer

West Virginia Section: The twelfth meeting of the West Virginia Section was held at Huntington on September 13–14, with headquarters at the Prichard Hotel. The meeting was well attended, the registration of 154 being well above the average for the past several years.

The first technical session opened Wednesday morning with Chairman Max Jones presiding, as he did at all other sessions. Mayor W. W. Payne of Huntington was unable to appear for the scheduled address of welcome, and time was thus available for the reading of a nonscheduled paper by W. A. Welch, Industrial Chemical Sales Div., West Virginia Pulp & Paper Co., Philadelphia, Pa., entitled "Factors Affecting the Efficiency of Taste and Odor Removal by Activated Carbon."

The regular schedule was then begun with "Questions and Answers on Softening Operations at Princeton" with Nick Leshkow, superintendent of the West Virginia Water Service Co., Princeton; and Wallace Grant and John B. Douglass, respectively chemist and construction engineer for the same company at Charleston, participating. The next speaker, E. E. Chandler, superintendent of the Williamson Water Dept., described an experience in "Cleaning Filter Media With Inhibited Hydrochloric Acid." A commercial organization undertook to clean filter sand in place at Wil-

(Continued on page 88)

(Continued from page 86)

liamson using the acid method. After using $750 worth of acid in the partial cleaning of one 0.5-mgd. filter, the firm asked to be relieved of its contract. A. R. Todd, superintendent of filtration at Wheeling, next gave an account of "The Effect of Gob Fills and Acid Coal Mine Drainage on the Manganese Content of Surface Streams and Underground Waters." Cecil Coffield, water commissioner at Parkersburg, next read a paper on the "Manganese Problem at Parkersburg." Two collector-type wells in that community yield a water with a high manganese content, necessitating special treatment.

The program scheduled for the afternoon of September 13 deserves special comment. The group first visited the Huntington filtration plant and observed practical demonstrations and exhibits of the versenate hardness test, filter sand analysis, coagulation study, and such equipment as an electric titrimeter, spectrophotometer, pipe locator, leak locator and cutting torch. After two hours at the filter plant the group visited the C. I. Thornburg Co. to view an elaborate display of water works tools, supplies and equipment, and to observe such demonstrations as the freezing of a service line to permit repairs under pressure, tapping of a main under pressure, pipe bending and threading, joint pouring and pipe pushing. Refreshments were served to the group.

The second day of the meeting opened with a showing of the new film, "The Dorr Way" presented by Roy Dragone, Dorr Co., New York. This was followed by an address by A.W.W.A. Secretary Harry E. Jordan, who chose for his subject the question "Is There a National Water Problem?" Pointing out that sufficient water is provided by nature for present and future needs of this country, he observed that it is up to man to use his brains to have this water available when and where he needs it.

The topic "Water Works Disasters in West Virginia" was developed in a panel discussion headed by H. K. Gidley, director of the Div. of San. Eng., State Dept. of Health. Alex Fischback, district engineer for the U.S. Geological Survey at Charleston, described the destructive flash floods which are not uncommon in West Virginia and illustrated his remarks with a series of slides. P. W. Tingley, maintenance engineer with the West Virginia Water Service Co. at Charleston, gave practical advice on the restoration of plant equipment which has been inundated by flood waters. W. E. Holy, sanitary engineer with the U.S. Public Health Service, Washington, D.C., presented a timely discussion of "Contamination of Water Supplies by Radioactive Wastes."

The final scheduled address was presented by W. S. Staub, vice president of the West Virginia Water Service Co., Charleston, who spoke on "The Small Water Works Plant Operator's Place in His Community." As an added attraction, primarily to aid the "one-man" water works, the Section and the Public Service Commission sponsored a half-day short

(Continued on page 90)

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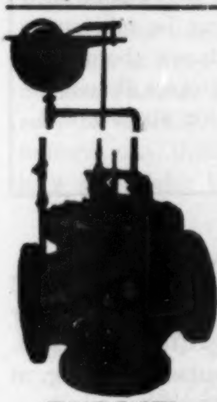

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Preventing Stream Pollution From Oil Pipeline Breaks: A guidebook of recommended practice. *Ohio River Valley Water Sanitation Commission, 414 Walnut St., Cincinnati 2, Ohio (1950) Available on request from Edward J. Cleary, executive director and chief engineer*

The first of a series of guidebooks being developed by the Ohio commission, this booklet discusses a problem that has become of increasing importance as the grid of oil pipelines has been extended throughout the country. Attention is focused on methods of preventing released oil from reaching waterways and causing pollution. Impounding, burning and spraying with special compounds are among the methods discussed. Proper installation methods to minimize or prevent pipeline breaks are also considered.

Statics and Strength of Materials. *Jasper O. Draffin & W. Leighton Collins. Ronald Press Co., New York (1950) $6.50*

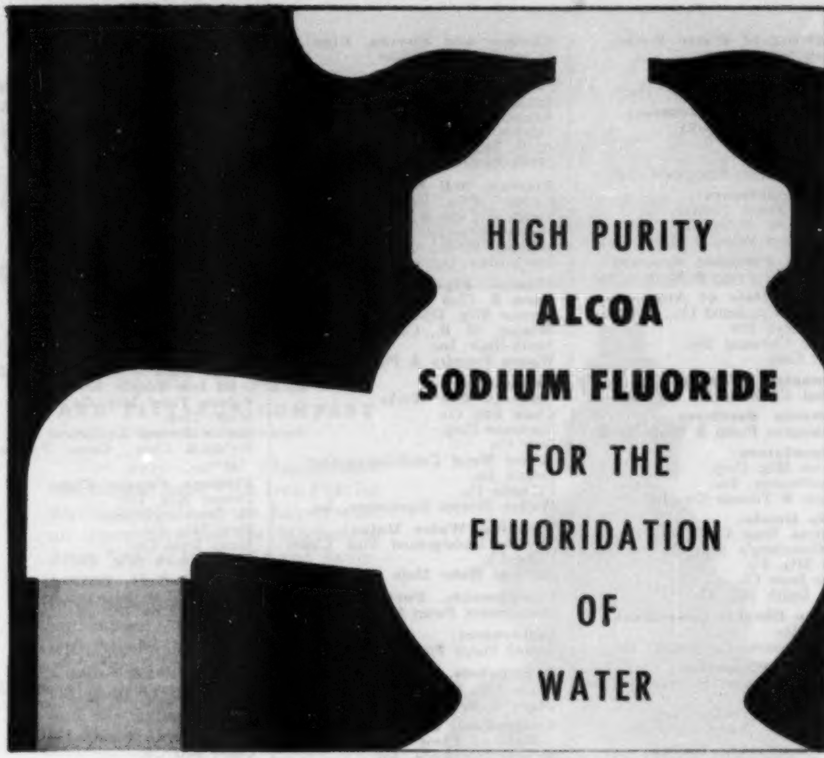
The authors have designed this textbook to meet the needs of those who have a mathematical background that goes up to but does not include the calculus, and who want fundamental information on the permissible stresses to which various materials can be subjected under relatively uncomplicated conditions. Although not, therefore, an advanced work, it is hardly light reading. Among topics discussed are coplanar force systems, friction, centroids, working stresses and welded joints.

(Continued from page 88)

school on "Instruction and Demonstration of Water Meter Testing and Repairing." The school was held in the Huntington Water Corp. meter shop on the afternoon of September 14, and was attended by 52 persons.

The meeting was concluded with a banquet on Thursday evening, at which J. Hanly Morgan of Huntington presided as toastmaster.

H. K. Gidley
Secretary-Treasurer



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Softeners:
Cochrane Corp.
Dearborn Chemical Co.
Dorr Co.
Graver Water Conditioning Co.
Hungerford & Terry, Inc.
Infilco, Inc.
Permutit Co.
Roberts Filter Mfg. Co.
Walker Process Equipment, Inc.

Softening Chemicals and Compounds:
Calgon, Inc.
Infilco, Inc.
Permutit Co.
Tennessee Corp.

Standpipes, Steel:
Chicago Bridge & Iron Co.
Pittsburgh-Des Moines Steel Co.

Steel Plate Construction:
Bethlehem Steel Co.
Chicago Bridge & Iron Co.
Pittsburgh-Des Moines Steel Co.

Stops, Curb and Corporation:
Hays Mfg. Co.
James Jones Co.
A. P. Smith Mfg. Co.

Storage Tanks; see Tanks

Strainers, Suction:
M. Greenberg's Sons
R. D. Wood Co.

Surface Wash Equipment:
Permutit Co.
Stuart Corp.

Swimming Pool Sterilization:
Everson Mfg. Corp.
Omega Machine Co. (Div., Builders Iron Fdry.)
Proportioneers, Inc.
Wallace & Tiernan Co., Inc.
Welsbach Corp., Ozone Processes Div.

Tanks, Steel:
Bethlehem Steel Co.
Chicago Bridge & Iron Co.
Pittsburgh-Des Moines Steel Co.

Tapping Machines:
Hays Mfg. Co.
A. P. Smith Mfg. Co.

Taste and Odor Removal:
Cochrane Corp.
Industrial Chemical Sales Div.
Infilco, Inc.
Proportioneers, Inc.
Walker Process Equipment, Inc.
Wallace & Tiernan Co., Inc.
Welsbach Corp., Ozone Processes Div.

Telemeters, Level, Pump Control, Rate of Flow, Gate Position, etc.:
Builders-Providence, Inc.

Turbidimetric Apparatus (For Turbidity and Sulfate Determinations):
Hellige, Inc.
Wallace & Tiernan Co., Inc.

Turbines, Steam:
DeLaval Steam Turbine Co.

Turbines, Water:
DeLaval Steam Turbine Co.

Valve Boxes:
Central Foundry Co.
James B. Clow & Sons
Ford Meter Box Co.
M & H Valve & Fittings Co.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.
R. D. Wood Co.

Valve-Inserting Machines:
A. P. Smith Mfg. Co.

Valves, Altitude:
Golden-Anderson Valve Specialty Co.
Ross Valve Mfg. Co., Inc.

Valves, Butterfly, Check, Flap, Foot, Hose, Mud and Plug:
James B. Clow & Sons
M. Greenberg's Sons
M & H Valve & Fittings Co.
Rensselaer Valve Co.
R. D. Wood Co.

Valves, Detector Check:
Hersey Mfg. Co.

Valves, Electrically Operated:
James B. Clow & Sons
Golden-Anderson Valve Specialty Co.
Kennedy Valve Mfg. Co.
M & H Valve & Fittings Co.
Philadelphia Gear Works, Inc.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.

Valves, Float:
Golden-Anderson Valve Specialty Co.
Ross Valve Mfg. Co., Inc.

Valves, Gate:
Dresser Mfg. Div.
James Jones Co.
Kennedy Valve Mfg. Co.
Ludlow Valve Mfg. Co.
M & H Valve & Fittings Co.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.
R. D. Wood Co.

Valves, Hydraulically Operated:
James B. Clow & Sons
Golden-Anderson Valve Specialty Co.
Kennedy Valve Mfg. Co.
M & H Valve & Fittings Co.
Philadelphia Gear Works, Inc.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.
R. D. Wood Co.

Valves, Large Diameter:
James B. Clow & Sons
Kennedy Valve Mfg. Co.
Ludlow Valve Mfg. Co.
M & H Valve & Fittings Co.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.
R. D. Wood Co.

Valves, Regulating:
Golden-Anderson Valve Specialty Co.
Ross Valve Mfg. Co.

Valves, Swing Check:
James B. Clow & Sons
Golden-Anderson Valve Specialty Co.
M. Greenberg's Sons
M & H Valve & Fittings Co.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.
R. D. Wood Co.

Waterproofing
Dearborn Chemical Co.
Inertol Co., Inc.

Water Softening Plants; see Softeners

Water Supply Contractors:
Layne & Bowler, Inc.

Water Testing Apparatus:
Hellige, Inc.
Wallace & Tiernan Co., Inc.

Water Treatment Plants:
American Well Works
Chain Belt Co.
Chicago Bridge & Iron Co.
Dearborn Chemical Co.
Dorr Co.
Everson Mfg. Corp.
Graver Water Conditioning Co.
Hungerford & Terry, Inc.
Infilco, Inc.
Pittsburgh-Des Moines Steel Co.
Roberts Filter Mfg. Co.
Stuart Corp.
Walker Process Equipment, Inc.
Wallace & Tiernan Co., Inc.
Welsbach Corp., Ozone Processes Div.

Well Acidizing:
Dowell Incorporated

Well Drilling Contractors:
Layne & Bowler, Inc.

Wrenches, Ratchet:
Dresser Mfg. Div.

Zeolite; see Ion Exchange Materials

A complete Buyers' Guide to all water works products and services offered by A.W.W.A. Associate Members appears in the 1948 Membership Directory.

"Look, Bill, what the boss just gave me!"

BILL: *What d'ya mean, the boss gave you? The boss never gave anybody one of his favorite cigars.*

BOB: *It's this way. I happen to walk into the boss's office about something and there he is, pounding the desk at Joe because all the electrical conduit in the pump galleries has got to be replaced. Seems it's the second time in two years they've had to do it.*

BILL: *Well! What's he expect? Corrosion would lick anything in those galleries in two years.*

BOB: *That's what Joe said.*

BILL: *And what's all that got to do with that cigar?*

BOB: *Because just then I said why didn't they use conduit made of Everdur. Seems the Everdur we used for all our valve stems has been in so long they'd completely forgotten all about it.*

BILL: *Hmm. I see. Matter of fact, I'm surprised the boss didn't give you two cigars.*

Wherever corrosion is a problem, that's the place to consider EVERDUR*, as sewage and waterworks engineers everywhere can tell you.

This group of ANACONDA Copper-Silicon Alloys has long been foremost among corrosion-resisting metals—both for performance and for ease of fabrication. Everdur is available in all wrought commercial shapes and in casting ingots. It is readily welded.

If you need a tough, corrosion-resistant metal, let us show you how EVERDUR can help. The American Brass Company, Waterbury 20, Connecticut. In Canada, Anaconda American Brass Ltd., New Toronto, Ont.

*Reg. U. S. Pat. Off. 50150

Where corrosion resistance counts—consider Everdur ANACONDA®
Copper-Silicon Alloys